DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

THE

BULLETIN

OF THE

BEACH EROSION BOARD

OFFICE, CHIEF OF ENGINEERS
WASHINGTON, D.C.

VOL. 10 (compl. JULY 1956 NO. 1

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WIND DISTRIBUTION OVER SEA WAVES

by

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This paper appeared originally in the German language, in the German periodical Naturwissenschaften in 1948 (Vol. 35, pp. 230-234), under the title "Das Windfeld uber den Meereswellen". It has been translated at the Wave Research Laboratory, Institute of Engineering Research, University of California, by Mr. J. Kukk, and the translated version edited by Messrs. Ovald Sibul and R. L. Wiegel, research personnel at that institution.

Developments of modern physics have presented to us knowledge of atomic relationships which we could not perceive without special aids. Because of this, an almost paradoxical situation has resulted; we are often better informed about such "hidden" things than about more obvious everyday natural occurrences. For example, we need only to think of our present inability to predict accurately the progression of weather, for even in those branches of the natural sciences for which sufficient mathematical systems of formulas have been developed, as is the case in wave theory, our knowledge of the physical relationships is still very limited. We know, to be sure, that the wind generates the waves, however, we do not yet clearly understand the mechanism of this transfer of energy. There are four conceivable ways in which turbulent wind flow could produce and develop water surface waves, and three of these have become bases for well-developed theories.

Lord Kelvin(1)* and v. Helmholtz(2) proceeded from the assumption that a compression of stream lines of the wind over the wave crests produced low pressure areas, and the fanning out in the wave troughs produced high pressure areas. The suction and compression forces resulting from this amplify any insignificant disturbances of the water surface to a point where the differences in pressure are equalized by the forces of gravity. The equations for waves developed from these assumptions, even when considering surface tension, are contradictory to experience in respect to the required minimum velocity of 670 cm/sec for the generation of the slowest waves, as these waves actually occur at considerably smaller wind velocities. Moreover, the existence of such a limiting wind velocity is debatable.

Later, Jeffreys(3) considered the wind pressure on the windward side of the wave to be important in the development of the wave. He equates the work done by the wind pressure on the wave to the energy consumed in the water by friction in the wave motion, and obtains expressions for wave values in which there still exists an unknown constant, so that a verification of the theory is not possible at present.

Jeffreys also investigated the problem of whether the tangential shearing force of the wind on the water surface - which in the case of a

* Numbers in parentheses refer to references listed at end of article.

string could cause vibrations - would suffice as an explanation of sea waves. He obtains, however, results (minimum velocity 480 cm/sec) which do not agree with experience.

Finally Seilkopf[4], more qualitatively, developed views on the generation of waves; he presented the opinion that above all, the vertical motions within the air were determining factors. According to Seilkopf the wave motion is weakened and continuously influenced by turbulent bodies of air which hit the water surface under slight, yet noticeable angles of incidence, and at the same time disperse. As proof for his views, Seilkopf introduces the known differences in wave characteristics for cold and for warm air. While in a turbulent cold air the bodies of air moved downward from a greater height directly onto the water surface and rapidly generate a series of steep waves; smoother and more regular wave conditions develop in warm air, as then there is a rolling and whirling motion of a thin layer of cold air immediately adjacent to the water surface.

The abovementioned theories have only one thing in common; they do not agree with the scant measurements that have been made, if a check on them is possible at all. Perhaps the question of waves should not be approached from a one-sided viewpoint such as suction, or wind pressure, or friction, or turbulence, before clarification is made as to what extent each of the four causes, each surely effective in its own way, contributes to the generation and further development of the waves. Progress in this field can be expected only when new and comprehensive measurements are made, as theory has advanced far ahead of empiricism.

Since the generation and propagation of waves represent a typical boundary problem between oceanography and meteorology, one can approach the question from two sides. While oceanography strives to obtain measurements of the shape and motion of waves, as well as of the forces involved; meteorology endeavors to study the field of wind over waves. This type of observation might well make an essential contribution to the understanding and solution of the problem, since the changes of the air flow over a wave-covered water surface -- especially the accompanying frictional losses -- in combination with simultaneous wave measurements, will most likely provide information about the manner by which wind affects the water surface. Since waves are both moving and very irregular in shape, it has not yet been possible to accurately establish the stream lines of wind flow over a wave. One must be content with obtaining average wind measurements, made with cup anemometers simultaneously at various heights above the water surface. Such measurements are relatively difficult to obtain at sea, which explains the lack of data. Over deep water, wind measurements can be undertaken only from floating bodies; these participate to greater or lesser extents themselves in the wave motion, depending upon the relation of their dimensions to the wave length. Because of this, oscillations of the wind gage are inescapable and inaccuracies in the knowledge of elevations at which the measurements are taken are inevitable; as these are unknown quantities it has not been possible to compensate for them.

Because of these technical difficulties in making measurements at sea, the investigations with solid wave models which Motzfeld[5] employed in a wind channel in order to study the flow over such wavy walls are of much significance. One must keep in mind, however, that the experimental conditions deviate essentially from conditions of waves at sea, such as wave progression, yielding of water surface, and orbital motion of water particles. Therefore one cannot expect that the results obtained by Motzfeld with solid wave models would also apply to the sea.

Before describing the investigations by Motzfeld, a few important basic concepts of the theory of turbulence, which will appear below over and over again, will be briefly explained. In observing flow along a wall we are accustomed to describe the internal bonds caused by turbulence between adjacent stream lines as the effect of tangential shear force, or a tangential shear stress τ, (force/unit area). In the air layer close to the ground this shear stress can be considered as a good approximation, independent of height, and thus can be set equal to the shear stress τ_0, acting immediately at the ground, or as the case may be, at the water surface. Still more frequently than τ_0 in the theoretical representation of turbulent flow, there appears the following combination of shear stress τ_0, and air-density, ρ; $\sqrt{\tau_0/\rho}$. Since the value is essentially determined by the shear stress, and because it has the dimension of a velocity, it is denoted as shear stress velocity u^*. It has an order of magnitude of the turbulent fluctuations. Prandtl improved extraordinarily the description of turbulent processes in formulas by introducing the term mixing length. Analogous to the mean free path of molecules in the kinetic theory, the mixing length is defined as the length of the path of an air particle through which it travels in an isolated manner, retaining its temperature, its impulse, etc., before it again mixes itself with its surroundings. In the layer of air near the ground, this mixing length increases with height in a linear manner. Along a smooth surface it is equal to zero. However, in the case of rough surfaces, the mixing length at the wall, i.e., at the peaks of wall roughnesses, will not vanish, but will be determined by the dimensions of humps, or as the case may be, of concavities. Thus, at a rough wall the minimum value of the mixing length can be expressed by the formulas through a value z_0, which will be called the roughness, and which in a simple manner is dependent upon the humps or concavities in the wall. However, this obvious meaning of roughness value z_0, when considering the vertical profile of wind velocity (where z_0 has an important part), will have to give way to a more formal one. According to Prandtl[6], the vertical wind velocity profile in the layer of air adjacent to the ground (marked by a shear stress independent of elevation and linear increase in mixing length) can be expressed with sufficient accuracy by the logarithmic formula:

$$u = \frac{u^*}{K} \ln \frac{z + z_0}{z_0} = 5.75\, u^* \log \frac{z + z_0}{z_0} \qquad (1)$$

where:

u = the wind velocity,
z = the elevation above the ground,

$u^* = \sqrt{\tau_0/\rho}$, the shear stress velocity,
τ_0 = the shear stress
ρ = the density
K = 0.4 - the universal constant of turbulence
z_o = the roughness height, (cm),
ln = the logarithm to the base e, and log is the logarithm to to the base ten.

Over rough walls, the value z_o is directly related to the measurements of wall-roughness. Thus, Prandtl gives, for sand roughness, $z_o = k/30$, where k represents the average diameter of the sand grains glued on it.

For flow over smooth walls, the transition between the smooth wall and the turbulent air flow is not abrupt, but rather is tempered by a laminar boundary layer of a thickness of approximately 1 mm, and there exists according to v. Karman(7) a somewhat modified form of the logarithmic law of wind velocity, which however could be transformed into Equation (1) if we defined roughness value z_o, which is proportional to the inverse of the shear stress velocity u^* ($z_o = 1/u^*$). These roughness values are several orders of magnitude smaller than the usual ones for rough walls.

Motzfeld investigated in a wind channel the flow over various wave shapes (sine, trochoid, and waves with sharp crests) and drew the following conclusions.

Over wave profiles with sharp crests the flow separates from the boundary at the crests; eddies are formed at the leeward side of the wave. The average vertical distribution of wind velocity is represented by the equation:

$$u = 5.75\, u^* \log (7.25\, z/h); \quad z_o = h/7.25 \qquad (2)$$

The roughness parameter, z_o, is proportional to the wave height h. We observe thus a relationship similar to the case of rough walls.

At wave profiles with round crests (sine, trochoid) Motzfeld found no separation of the flow; rather there resulted an average distribution of wind velocity, which is similar to that of a smooth surface

$$u = 5.75\, u^* \log (z/z_o), \text{ where } z_o = \frac{\nu}{3u^*}\, 10^{3.17 \frac{\tan \alpha_m}{u^*}} \qquad (3)$$

wherein ν is the kinematic viscosity of the air and $\tan \alpha_m$ the maximum slope of the wave profile. Since the variation of the exponent remains small, the roughness z_o is essentially inversely proportional to the shear stress velocity u^*, where the coefficient is dependent upon the wave form ($\tan \alpha_m$). To be sure, this influence of the wave form is relatively small.

This result permits the conclusion that for wave forms with round crests the frictional component of the flow resistance (caused by tangential

shearing force) is greater than the component of pressure. This conclusion is confirmed by the coefficient of pressure and friction resistances, c_d and c_r, as determined by Motzfeld. For wave profiles with sharp crests, the coefficient c_d of the pressure resistance amounts to approximately 87 to 88 percent of the total coefficient of resistance. The effect of pressure for wave profiles with round crests decreases considerably in comparison to the effect of friction. Thus the coefficient c_d of the pressure resistance for sine, or as the case may be, trochoid profiles, with a ratio of wave height/wave length = 0.1 (steep profiles) amounts to 39 to 44 percent of the total value, and reduces finally to 18 percent for a profile corresponding to a sea wave with regard to steepness (wave height/wave length = 0.05). From the experiments on models made by Motzfeld it seems to follow that for waves with sharp crests the pressure resistance is the controlling factor, while for round wave profiles, the friction resistance is the controlling factor.

We must now establish whether and to what extent the knowledge obtained by Motzfeld from his experiments with models holds for air flow over moving sea waves. Prior to this time there have been no suitable observations available. In the published measurements of wind velocities (Wust[8], Montgomery[9], Shoulejkin[10] and Bruch[11]) as well as in the wind gradient measurements obtained during the International Gulf Stream Investigation in 1938 aboard the ALTAIR, the data were taken within a few meters immediately above the water surface and wave measurements were not made simultaneously; therefore, the relationships found by Motzfeld for his wave models could not be accurately checked. This explains why Rossby[9] found a tendency for the roughness of the sea surface to decrease with increasing wind velocity, while Sverdrup[12] postulated an increase, and Model[13] derived a constant from the Bruch measurements. The difficulty seemed to arise from the definition of the roughness parameter as z_o.

The author, under the sponsorship of the Maritime Division of the Office of Meteorology for North West Germany, Hamburg, has, among other things, made measurements of wind velocities in the air layer immediately adjacent to the water surface simultaneously with wave measurements. Because of lack of time and the difficulties inherent in such measurements over deep water, the investigations were undertaken over shallow water. Thus, there existed the possibility of mounting measuring instruments to a mast rigidly implanted in the bottom of the sea. These measurements were made in the tidal flats northwest of the island of Neuwerk during July and October 1947.

The advantages of this location, aside from the mentioned possibility for erection of the mast, consisted of a uniform depth of water of some 175 to 200 cm at high tide over a large area, and the relatively great regularity and thereby good measurability of the waves. A disadvantage was the considerable tidal motion with its associated variation in water depth.

The average wind velocity was measured simultaneously at 6 different

heights, between 4 and 200 cm above the wave crests, using cup anemometers. At the same time the following wave characteristics were measured; wave direction; wave height, h; wave period, T; and wave velocity, C (from which the wave length, λ , could be determined by use of the equation $\lambda = CT$). In addition, the depth of water and the tidal motion (direction and velocity) were measured. Temperature and humidity were also closely noted. Details of the measurements have been published elsewhere(14). These investigations made during 1947 (will be continued in 1948), have resulted in a total of 193 wind velocity-profiles (simultaneous wind measurements for 6 different elevations) with wave and current measurements, and 73 wind velocity profiles over the dry tidal flat.

In addition, special measurements were made of such things as wind profiles from 2 to 24 cm over the ripples of small shallow pools, such as remain at low tide on the otherwise dry tidal flats, and three dimensional wind profile measurements to investigate the change of wind profile with the change of base (land-sea, land-tidal flat).

Before evaluating these data the influence of tides was eliminated by plotting all wind velocity measurements on a coordinate system relative to the water surface. The individual tide-corrected wind profiles were then summed up into average wind profiles in order to reduce the influence of scatter. These were, without exception, in conformity with the logarighmic formulas and from these, the respective values for roughness, z_o, and shear stress velocity, u*, were determined using the method of least squares.

Since we wished to establish whether the proportionality between roughness and wave height for sharp crested waves as found by Motzfeld was also valid for traveling sea waves, wave heights were first used as the organizing principle for summarizing individual runs into average values. Since the depth of water varied between 1 and 175 cm in our measurements, the wave heights also covered a relatively wide band from 0.1 to 30 cm. It was necessary to apply a correction to the elevations of the wind gages, as the datum had been the plane of the crest of waves. A new datum, the mean water level, was arrived at by adding a half of the average wave height. The determination of the roughness parameter for the various ranges of wave heights revealed no relationship between roughness and wave height. The relation(2) found by Motzfeld with models of sharp crested waves did not seem to be substantiated, at least for shallow water waves.

Next an investigation was made as to whether the distribution of wind velocities(3), as derived from model experiments by Motzfeld on waves with round crests, were applicable for sea waves. For this the individual wind profiles had to be arranged according to shear stress velocities, u*, (that is, practically according to the wind velocities) and within these limits according to wave form ($\tan \alpha_m$).

The correct value for the maximum slope of the wave profile could only be derived if the exact slope of the wave were known. This, however,

was not the case. Lacking a more suitable measured value, the steepness of the waves, i.e., the ratio of wave height to wave length, h/λ, had to serve as a provisional measure for the maximum slope of the wave profile. In the case of sine waves, $\tan \alpha_m = \pi h/\lambda$ holds. Here too the elevation measurements were corrected according to the average wave height.

As a result of this work, it was found that there was a definite dependence of roughness upon shear stress velocity in the sense called for by Equation (3). An influence of wave steepness, however, could not be established, or perhaps was obscured by the scatter.

In order to cover the average values as often as possible, grouping according to the steepness, h/λ, was discontinued, and only an arrangement according to the wind velocities was made. The number of individual runs in the six different wind groups lay somewhere between 20 and 25. The average wind profiles obtained in this manner have been plotted in Figure 1. In this figure a logarithmic scale was selected for the elevations $z + h/2$. It can be noted that the vertical distributions of wind velocities are represented by straight lines, hence the logarithmic law (1) is satisfied. Figure 2 gives the roughness values, z_o, as obtained from these profiles (also plotted on a logarithmic scale) as a function of the proper shear stress velocity, u^*. This presentation also contains:

1. The roughness values z_o as a function of u^*, which, according to the formula by v. Karman, would result for flow over a smooth surface.

2. The roughness values z_o as a function of u^*, which, according to Equation (3) by Motzfeld, would hold for a flow over waves with round crests. Here a sine wave ($\tan \alpha_m = \pi h/\lambda$) was assumed which had a steepness $h/\lambda = 0.05$, which corresponds to the average condition on the sea.

The measurements show that the roughness values for the sea surface indicate a slight decrease with increasing shear stress velocity (i.e., wind velocity), and are parallel to the curve for smooth surfaces given by v. Karman (shifted toward higher roughness). The numerical values for z_o lie between 0.006 and 0.002 cm. The values computed from Equation (3) by Motzfeld come remarkably close to the measured ones. Differences could be ascribed to the deviation of the waves from simple sine form, which was used as the basis for computation. The fluctuations in the measured roughness values probably represent the effects of scatter; they are not produced by changes in steepness. The measured values of z_o could be approximated by the relation $z_o = \dfrac{\nu}{2.1\, u^*}$; hence, the vertical increase in wind velocity over tidal flats takes on the form

$$u = 5.75\, u^* \log \frac{2.1\; u^*}{\nu} (z + h/2). \qquad (4)$$

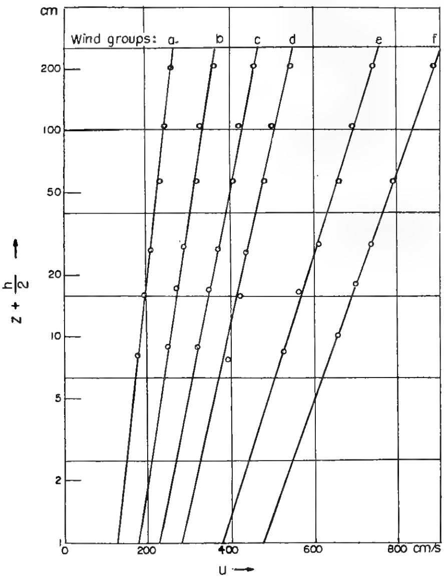

Fig. 1

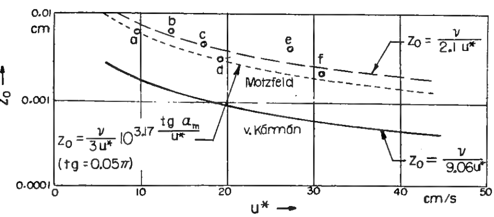

Fig. 2

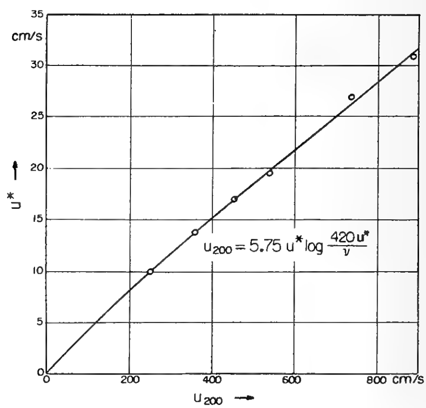

Fig. 3

FIGURES 1-3

For a constant measurement elevation, e.g., $z + h/2 = 200$, Equation (4) results in a relationship between the wind velocity u_{200} for that elevation and the shear stress velocity, u^* (Figure 3). This representation reveals that the shear stress velocity is related in a linear manner to the wind velocity. We are now in the position to determine the shear velocity u^*, with the help of Figure 3, from a single wind velocity measurement, say for example, at an elevation of 200 cm, and thus according to Equation (4) to compute the total vertical wind profile.

Wind profile measurements above the waves of shallow pools at low tide lead to roughness parameters of an order of magnitude equal to that for tidal sea providing that an initial time period of 2 to 3 seconds has elapsed after the change of base.

The following conclusions have been made from the studies of these measurements over the tidal sea. Wind profile over the tidal sea, at least from the surface to an elevation of 2 m, is not influenced by wave height. The sea surface thus does not possess any constant "roughness" in a real sense. On the contrary, the average field of wind over the sea waves could be satisfactorily represented by equations which are similar to v. Karman's formula for flow over smooth surfaces, and which are characterized by the inverse proportionality between the roughness z_o - here purely formally defined - and the shear stress velocity u^*. The so-called "roughness" of the water surface thus decreases with increasing shear stress velocity (i.e. with increasing wind velocity) independent from wave height. Whether the slight influence of the wave shape ($\tan \alpha_m$) as found by Motzfeld in his model experiments actually exists will have to be left unanswered, as it could not be determined from the measurements, possibly having been obscured by scatter.

If we now return to the point of departure of our observations, the question of transformation of energy from wind to waves, we would conclude on the basis of our measurements that as indicated by the experiments on models by Motzfeld, the pressure resistance of sea-waves plays a minor role in comparison to the frictional resistance. In wave theory, then, the term describing the dependence upon wind pressure would have to be less than the term describing the tangential force of friction. Above all, it appears that during the generation of initial waves with an approximate wave length of 2 cm, friction is the most important criteria next to surface tension, while during the following transformation and further development of the waves, suction and pressure effects of the field of wind become important factors. Investigations striving for clarification of the question are still in progress.

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BEACH EROSION AT DURBAN, SOUTH AFRICA

The following article is based on correspondence from Colonel David E. Paterson, Beach Consultant for the City Council of Durban, Natal, South Africa. The problem at Durban, namely that of intensified shore erosion downdrift from harbor entrance structures, is similar to many problems in the United States. Description of the problem and efforts by local people to alleviate it (which include operation of a sand bypassing plant) is therefore considered to be of interest to readers of the Bulletin. Photograpsh used for illustration were furnished by Colonel Paterson.

Description of Problem

Durban, a port city in the Province of Natal, Union of South Africa, is located on the southeast coast of Africa. (Figure 1) This stretch of coast is directly exposed from about east-northeast through south to the open water of the Indian Ocean. Durban, as one of Southern Africa's leading year-round resort areas, is vitally concerned with the preservation of its beach and shore recreational area. The erosion problem at Durban started over 50 years ago, or sometime after construction of breakwaters at the entrance to the harbor. (See Figure 2.)

Construction of the south breakwater was commenced in 1882 and completed, along with construction of the north pier, shortly afterward. Intensive dredging at the harbor entrance was initiated in 1896, and since that time navigable channel depths of 42 feet below low water of spring tides have been maintained by dredging. As littoral drift is fairly heavy in this area and is predominantly from the southwest toward the northeast, considerable dredging has been required. An estimated 60,000,000 cubic yards of sand was removed from the vicinity of the harbor entrance through 1953, a portion of this dredged yardage being part of the original harbor deepening and the rest being maintenance. Over a 40-year span within this period, more than 40 acres of foreshore area were lost by erosion in the area north of the north pier to West Street.

Just north of West Street, improvements at the Lower Marine Parade project somewhat seaward of the alignment of adjacent shores. A rubble mound revetment was constructed to maintain the position of the shore line in this section. This revetment has required constant maintenance.

Past Corrective Measures

During the period 1938-1947 over 5 million cubic yards of sand were pumped from the harbor entrance area to the beaches north of the breakwaters. The material was dredged by the hopper dredge "Blesbok" (Figure 3) and pumped from that dredge, moored alongside the north pier, through a 42-inch discharge line to a point on the beach about 5,000 feet north of the north pier. The results of this 9-year replenishment operation were the restoration of about 34 areas of beach area between the north pier and West Street and about 40 additional acres of accretion in areas north of the revetted Lower Marine Parade to Umgeni River. The beach fronting the revetted area of the Marine Parade did not widen significantly, although accretion was occurring in areas on either side. Beach areas south of the pipeline discharge location continued to erode.

In 1950 a fixed bypassing plant was placed in operation. This plant consisted of a 16-inch pump with crane-supported suction mounted on a steel pier south of the south breakwater (see Figure 4). The pump was connected by flexible rubber coupling to a 16-inch discharge line which ran under the harbor entrance. When uncoupled from the discharge line the crane and pump suction could be moved along the pier to a protected location as required. This plant could dredge to 20 feet below low water and the suction pipe could swing through 180 degrees with a radius of 36 feet. During the period 1950-1954 this plant bypassed approximately 200,000 cubic yards of sand per year across the harbor entrance, discharging 5,000 feet north of the north pier until 1953, when the discharge point was shifted to a location just clear of the base of the north pier. (See Figure 5.) Replenishment at that rate apparently maintained stability of the shore line north of the harbor entrance to Umgeni River. However, damage still occurred during the more severe storms in the Lower Marine Parade area just north of West Street.

To create a wider beach in this latter area two groins were constructed in 1953 in addition to a storm drain (Somtseu) that acted as a groin to the north of this area. (See Figure 9.) The most southerly groin (number 1) widened the beach on its south side a maximum of 135 feet during the first six months after its construction in April 1953, but erosion caused by subsequent storms in September and October reduced the beach widening to about 85 feet.

Photographs of the revetted Lower Marine Parade area and the groins are shown in Figures 6 to 9.

Concluding Remarks

The City Council of Durban, on the basis that the city's operation of the bypassing plant materially assisted in maintenance of the harbor, requested assistance from the South African Railways and Harbours Administration in operation of the bypassing plant. In lieu of such assistance an agreement was reached whereby the Railways and Harbours

Administration will, with its existing floating plant, pump sand over the north pier to maintain the beaches at no cost to the city for a period of 18 years. Operation of the City's bypassing plant has been discontinued, being considered unnecessary as long as sand is supplied in connection with harbor maintenance.

Although it appears that a continuing supply of sand stabilizes the Durban shore, a protective beach of sufficient width in front of the Lower Marine Parade has not been obtained. Accordingly the City has extended groin number 1 and in February of 1955 was constructing another groin 900 feet to the south of that structure, in the belief that adequate protective beach width on this seaward section of shore could not be obtained by increased supply of material alone.

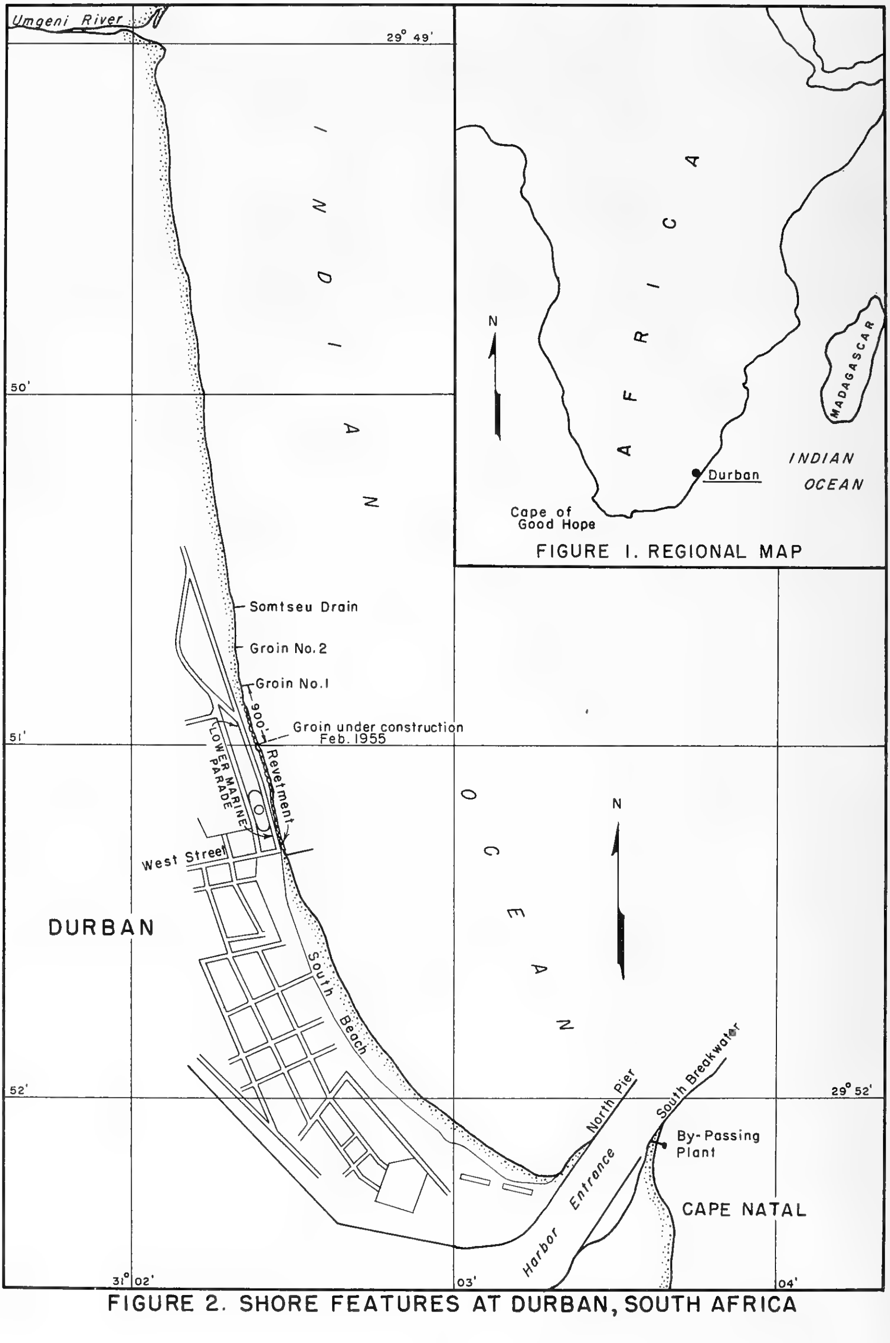

FIGURE 2. SHORE FEATURES AT DURBAN, SOUTH AFRICA

FIGURE 3 HOPPER DREDGE "BLESBOK" DISCHARGING OVER NORTH PIER THROUGH 42" PIPE LINE, JAN. 1938. ENTRANCE CHANNEL, DURBAN HARBOR

FIGURE 4 · SAND BY-PASSING PLANT, DURBAN, SOUTH AFRICA

23rd October 1953

FIGURE 5 · DISCHARGE FROM 16 IN. DIAMETER PIPE AT BASE OF NORTH PIER, DURBAN, SOUTH AFRICA

23rd October 1953
View Of South Beach Looking North

23rd October 1953
View Of Beach Looking South From West Street.
Bluff South Of Harbor In Background

FIGURE 6 · BEACH AT DURBAN, SOUTH AFRICA

23rd October 1953

Rubble Slope Protecting Lower Marine Parade.
Main Groin No. 1 In Background

23rd October 1953

Rubble Slope Protecting Lower Marine Parade

FIGURE 7 · RUBBLE-MOUND REVETMENT AT LOWER MARINE PARADE, DURBAN, SOUTH AFRICA

23rd October 1953
North View Of Main Groin No. I

23rd October 1953

Formation Of New Beach On South Side Of Groin No. I
Taken From Base Of Groin Looking South.
Rubble Protection Of Lower Marine Parade Shown In
Center Background.

23rd October 1953

Groin No. I In Far Background
Groin No. 2 Looking South

FIGURE 8 · GROINS AT DURBAN, SOUTH AFRICA

23rd October 1953
Groin No. 2 Top Of Somtseu Drain Just Showing In Distance.

23rd October 1953
Old Timber Groin With Groin No. 2 In Background And Somtseu Drain Projecting Seaward In Distance.

23rd October 1953
Somtseu Drain Acting As A Solid Groin

FIGURE 9 · GROIN NO. 2 AND SOMTSEU DRAIN, DURBAN, SOUTH AFRICA

SEDIMENT MOTION AT THE VICINITY OF A LITTORAL BARRIER

by
Ning Chien

This is a portion of a paper prepared by the author, at that time a graduate student research engineer at the University of California, under a research contract between the University of California (Institute of Engineering Research) and the Beach Erosion Board. The paper in entirety appeared under the same title in April 1955 as a Technical Report Series 14, Issue 17, of the Institute of Engineering Research, Wave Research Laboratory, College of Engineering, University of California.

INTRODUCTION

This investigation is a continuation of the study made in 1952 (Chien and Li) on the effect of a littoral barrier on a sandy coast. The littoral barrier is defined as any natural or man-made abrupt change in shore alignment, such as prominent head-lands, projecting rock cliffs, and coastal works like groins and jetties.

From the 1952 study, it was found that when the normal littoral drift is interrupted by the installation of a barrier, the rate of drift around the barrier will be resumed when the impounding basin of the barrier is filled by the sediment originating at the upcoast side. The conclusion thus drawn is true for the particular experimental arrangement employed in that study, and should by no means be considered as general. Indeed, the existence of natural littoral barriers indicates that there must exist certain means by which the sediment passes around the barriers without the formation of a beach. The purpose of this study is to investigate the possible mode of sediment transportation at the vicinity of a littoral barrier in laboratory scale.

This was achieved by reproducing a small reach of a natural barrier in the laboratory at which the motions of water particles and sediment was observed. In this way the means by which the sediment particles pass around the barrier are not observed directly, but rather deduced from their mode of transportation in front of the barrier. As will be shown later, the experiment was conducted in a model basin of limited size. The barrier was oriented at an angle with both the wave generator and the side-walls. Any waves reflected from the barrier are reflected again after reaching the flapper of the wave generator and side-walls, resulting in a very complicated wave pattern in the basin. For a successful operation of this type of experiment, it is imperative that the barrier must not reflect the approaching waves excessively. A preliminary study was therefore made to explore the possibilities in designing certain

types of structures which would function both as a barrier and as a wave absorber. The final technique adopted was to develop a model cliff in the wave basin by the scouring action of waves. While the overhanging cliff wall gradually takes its shape, a small submerged sand beach is developed at the foot of the cliff. The sediment transport along the beach is then measured, and the rate compared with that on a natural beach without the cliff. Finally, the scale effect of the model study is discussed in a qualitative manner, and a possible description of sediment motion around the littoral barrier is given.

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SEDIMENT MOTION ALONG A CLIFF

Field observations indicate that many of the natural littoral barriers have a rugged steep rock face, sometimes overhanging the sea, with a small submerged beach developed at the base of the cliff. Inasmuch as an artificial littoral barrier developed in a preceding series of tests did not function sufficiently well as a wave absorber, it appeared conceivable that a model cliff could be developed in a model basin by the action of waves themselves. This would call for cliff material which could be moved and re-shaped by the waves, but at the same time should have sufficient strength to stand up vertically against the action of the sea. After many trials it was found that a mixture of fine sand (median diameter 0.275 mm) and wet and swelled bentonite seemed to serve the purpose. Part of the bentonite was left in the mixture in small lumps which, after being exposed to the wave action, were more resistant against erosion than other parts of the mixture, thus giving an irregular surface to the exposed area.

The mixture was first molded into a thick vertical wall which was placed in a 1-foot by 6-foot by 12-foot wave basin at an angle of 20 degrees with the approaching waves (see Figure 1 (a)). A wave machine, driven by an A-C electric motor, was located at the opposite end of the basin. The speed of the wave machine, and consequently the period of the waves, could be adjusted by a Vickers speed reducer. The amplitude of the wave was controlled by an adjustable eccentric. A sand trap was installed at the downcoast end of the vertical barrier, and any sediment which settled at the bottom of the hopper was pumped continuously by a jet pump back to the upcoast end of the barrier through a return pipe (Chien, 1952). The jet flow was supplied by a sump pump which drew water from the basin near the wave machine. Throughout the experiment, a constant water depth of 0.53 foot was maintained in the basin, and the still-water level was 0.22 foot below the top of the barrier. A 0.52-second wave with an amplitude of 0.042 foot was chosen for the experiment.

When waves first hit the vertical wall most of the incident wave energy was reflected. In time the front face of the wall began to cave in. Clay particles were washed away and sand was left behind and accumulated at the base of the wall. Afterwards the sand piles were

remolded by the combined action of waves, littoral current and gravity, and a small submerged beach developed. By then the wave reflection was noticeably reduced, as more and more wave energy was absorbed by the sand beach and by the rugged surface of the overhanging wall. The run was stopped and the water drained. Figure 1 (b) shows a general view of the cliff model thus developed, which represents a small reach of a natural littoral barrier.

The motion of sediment along the submerged beach at the base of the cliff was next measured. In order to keep the size of the beach unchanged, the overhanging part of the cliff was first protected against further erosion by the waves. This was accomplished by pasting a layer of Hydrostone over the surface of the vertical wall (Figure 1 (c)). The wave basin was re-filled with water to its original level and the wave action resumed. The rate of littoral transport was determined by diverting the sediment-water mixture flow from the upcoast end of the return pipe into a container, and by measuring the sediment thus collected during a certain time interval. The measurement was repeated every 15 minutes until the rate of transport became constant. Figure 2 shows the profile of the cliff with a small submerged beach at its foot which is at equilibrium. The submerged beach starts at an elevation 0.32 foot from the still-water level. It has an overall slope of 2.2:10, except at the very end, where it inclined more or less at the angle of repose of the beach material. The width of the beach was about 0.63 foot. The littoral transport along such a submerged beach at the foot of a cliff was 1.60 lbs. per hour, and this is to be compared with the transport rate along the part of a natural sand beach which is submerged by the same depth of water.

Next the cliff model was replaced by a sand beach at a constant slope of 2.2:10 and oriented at 20 degrees with the approaching wave (Figure 4 (a)). The same sand which was mixed with the clay in the first part of the experiment was used. The run was repeated with the same characteristics of waves and the same depth of water in the basin. Figure 3 shows a profile of the beach when it finally reached equilibrium, and Figure 4 (b) is a general view of the beach. Along such a sand beach the littoral transport was 25.5 lbs. per hour. Again the upper part of the beach was stabilized down to an elevation which was 0.32 foot below the still-water surface, leaving the lower end of the beach open for further wave action (Figure 4 (c)). The sediment motion along the submerged sand beach was observed. In this condition the transport rate was reduced to a trivial 0.018 lb. per hour, indicating that there is practically no sediment movement along such a submerged beach when the cliff is not present. These two sets of experiments clearly demonstrate that the presence of a cliff considerably increases the littoral transport at larger depths, which, however, is still a small percentage of the total transport that can move along a normal sand beach.

a.

b.

c.

Sediment motion along a cliff: a. A thick vertical wall of sand-clay mixture was first molded in the basin at an angle of 20° with the approaching waves; b. the vertical wall was reshaped by the waves to form an overhanging cliff with a small sandy beach at the foot; and c. the cliff was stabilized by pasting its surface with a layer of hydrostone, and the motion of sedimen' along the submerged beach was measured.

FIGURE I

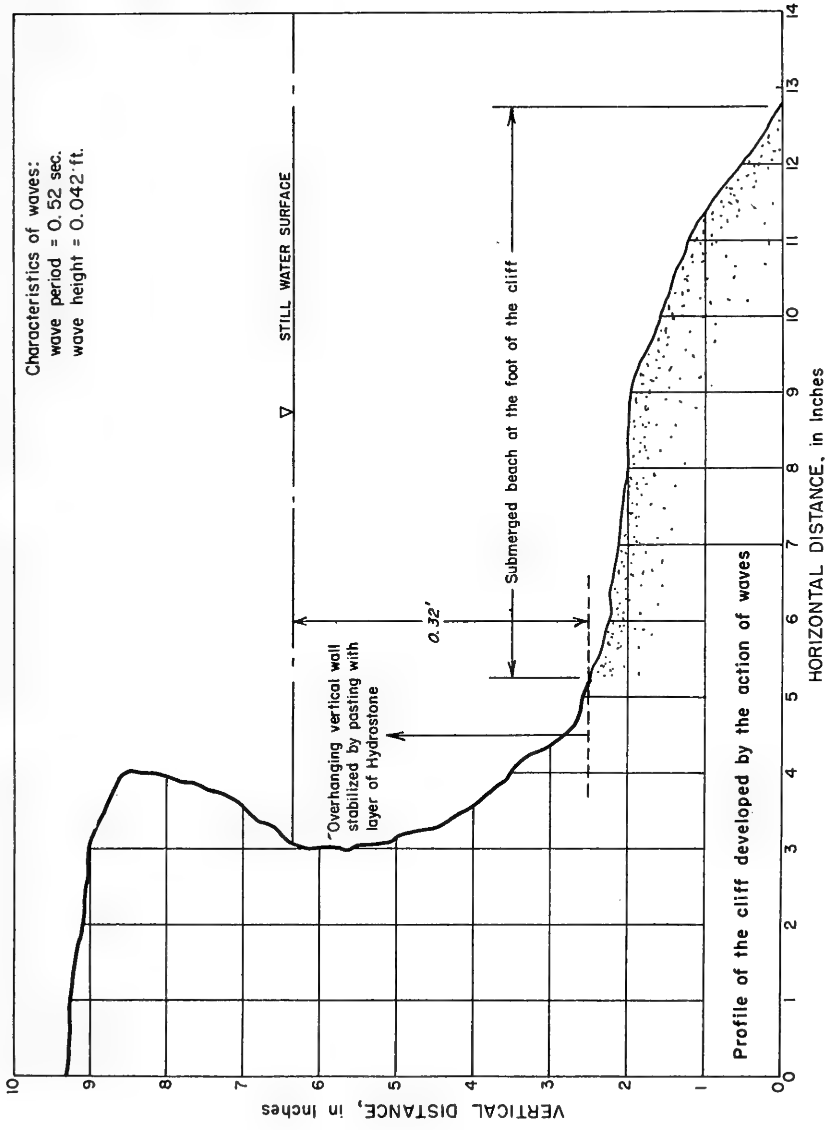

Characteristics of waves:
wave period = 0.52 sec.
wave height = 0.042 ft.
STILL WATER SURFACE
Submerged beach at the foot of the cliff
0.32'
Overhanging vertical wall stabilized by pasting with layer of Hydrostone
Profile of the cliff developed by the action of waves
HORIZONTAL DISTANCE, in Inches
VERTICAL DISTANCE, in Inches
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14

FIGURE 2

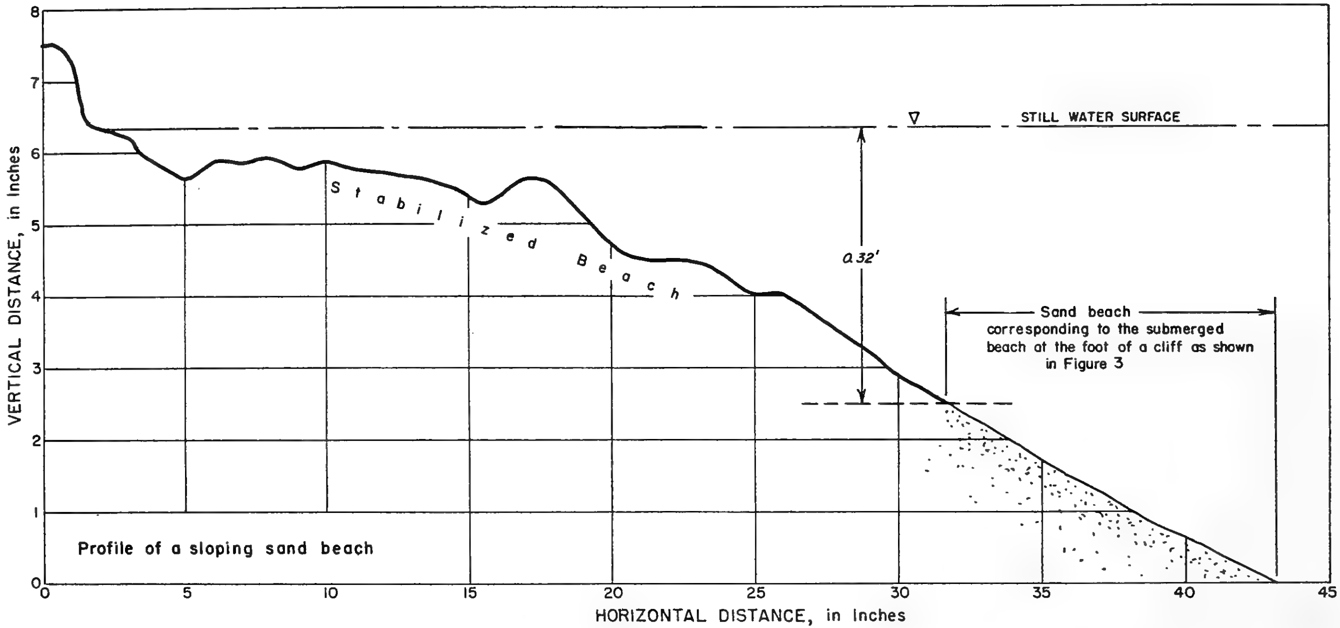
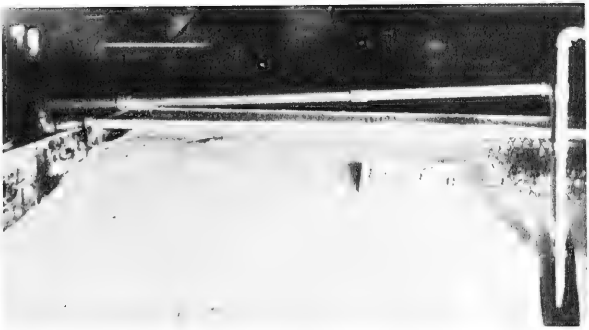

STILL WATER SURFACE
0.32'
Stabilized Beach
Sand beach corresponding to the submerged beach at the foot of a cliff as shown in Figure 3
Profile of a sloping sand beach
VERTICAL DISTANCE, in Inches
HORIZONTAL DISTANCE, in Inches
0
1
2
3
4
5
6
7
8
0
5
10
15
20
25
30
35
40
45

FIGURE 3



a.

b.

c.

Sediment motion along normal sand beach: a. A sloping sand beach oriented at an angle of 20° with the approaching waves; b. the same beach after modified by the wave action; and c. the stabilizing of the upper part of the beach with the lower end, which corresponds to the submerged beach at the foot of the cliff (Fig. 2), left open for further wave action.

FIGURE 4

SCALE EFFECT IN SUCH A MODEL STUDY

The question arises as to whether the ratio between the littoral transport along a model submerged beach at the foot of a littoral barrier, and that of a normal beach holds for the prototype. To successfully ascertain this, one must first determine how the sediment motion in these two cases is affected by flow turbulence, and the extent of scale effect on the generation, diffusion, and decay of turbulence. Owing to the complicated nature of the problem, the discussion is necessarily qualitative and even speculative.

When waves approach shoaling water, the wave crests tend to become steeper and narrower, while the troughs become flatter. Close to the shore line, waves begin to deform rapidly by increasing the front slope and reducing the rear slope, leading, in the limiting case of a breaking wave, to a front slope which goes beyond the vertical. The location of the breaking zone often coincides with that of the longshore bar. The tumbling over of wave crests creates local turbulence of high intensity which keeps much sediment in suspension to be carried downcoast by the littoral current. The suspension of sediment particles takes place in the zone where the turbulence is generated, and it is not associated with the turbulence which is in the process of diffusion and decay.

On the other hand, when a wave reaches a cliff part of it is reflected. The remaining part of the energy is converted into the kinetic energy of turbulent flow at the face of the cliff through impact. The turbulence thus generated gradually diffuses toward the sea, and in the meantime, decays through the action of fluid viscosity, until finally the two balance each other and the flow becomes irrotational again. In the process of expanding seaward, some of the turbulence reaches the neighborhood of the submerged beach at the base of the cliff, thereby keeping the local sediment in suspension if the strength of turbulence is large enough. The littoral transport in this case depends to a large degree on the extent of turbulent diffusion, and on the intensity of the diffused turbulent flow which is that left-over from the original turbulent energy after continuously being dissipated into heat.

Realizing that the suspension of sediment takes place in the turbulence-generation zone on a normal sloping beach, and in the turbulence diffusion zone at the foot of a cliff, one now turns to the question of what is the effect of the difference in scale in model and prototype on the different stages of turbulence development. In a flow problem which involves surface waves, the Froude law is used in conducting model studies. If l_r is the ratio of length scales and V_r is the ratio of velocity scales, then the Froude criterion states that

$$V_r = l_r^{\frac{1}{2}} \tag{1}$$

On the other hand, the turbulent flow pattern in the prototype and in the model will be similar if the Reynolds criterion is observed

$$\frac{V_r \; l_r}{\nu_r} = 1 \tag{2}$$

where ν_r is the ratio of kinematic viscosities of the prototype fluid and the model fluid. Combining Equations (1) and (2), one has

$$\nu_r = l_r^{3/2} \tag{3}$$

i.e., the prototype and model will be dynamically similar if the model fluid is so chosen that its viscosity observes Equation (3). Since water is used in the present model study, the result of the experiment therefore corresponds to the performance of an ocean of, say, heavy oil. One immediately sees that an ocean of water, and an ocean of oil cannot behave in a similar manner if the viscosity is an important factor in defining the flow pattern.

Fortunately, on a normal beach the breaking of the waves is largely due to an increase of wave amplitude, such that the wave form becomes unstable. The tumbling over of the wave front and its breaking into a large number of eddies are affected by the viscosity of the fluid only to a a minor extent. Since the motion of sediment takes place in the area where the eddies are created, the viscosity of fluid is a factor of secondary importance only. This is no longer true for the sediment motion along a submerged beach at the base of a cliff. It is common knowledge that the viscosity of fluid plays a significant role in dissipating turbulent energy into heat. In an ocean of water, the turbulence generated at the cliff may extend to a significant distance from the cliff before being completely destroyed by viscosity, while in an ocean of oil this distance would be very much limited. Naturally the sediment transport in the former will be larger than in the latter, due to the difference in the area of the submerged beach which is being exposed to the action of turbulence. The effect of viscosity on the diffusion of turbulence from a line source into an unlimited body of fluid is rather involved from the analytical point of view, although it can easily be studied experimentally in the laboratory. For instance, by shaking a plate under a large body of fluid, turbulence of controlled intensity can be generated over the entire area of the plate. The diffusion and decay of the turbulence thus created can then be studied by observing the trace of dye particles or concentration distribution of minute foreign particles. By using fluids of different viscosity, one can easily find out, at least qualitatively, what effect viscosity has on this type of turbulent diffusion.

From the discussion above, one may conclude that although the result of the model study indicates that the motion of sediment along a submerged beach at the base of a cliff is only a small percentage of the normal littoral transport on a sloping beach, it is conceivable that the two may

come much closer together in nature, considering the difference in scale between model and prototype.

EFFECT OF SECONDARY CURRENT ON SEDIMENT TRANSPORT

The modern theory on mass transport in water waves indicates that there exists a shoreward movement along the bottom and a movement at the surface, which may move either towards the sea or towards the land. The direction of surface motion and the relative strength between the surface and bottom movement depend on the parameter d/L, where d is the water depth and L is the wave length. This implies that at the front of a littoral barrier there exists a secondary flow which moves either in the clockwise direction, where the water particles move inward along the bottom, upward at the front of the barrier, and then outward at the surface as observed in this study, or in the counter-clockwise direction. When the roller moves in the clockwise direction there will be a slow upward movement of a large mass of water in the neighborhood of the barrier which is helpful in keeping sediment in suspension, in addition to the vertical fluctuating velocity of the turbulent flow. On the other hand, the clockwise motion of the secondary flow also keeps some of the turbulence generated at the cliff away from the beach, thereby reducing the strength of the turbulence prevailing at the beach surface. The reverse is true for the case of a counter-clockwise roller. The net effect of the secondary flow on the sediment motion therefore depends upon the relative strength of the turbulent flow and the secondary flow. No definite conclusion to this effect can be drawn at this moment.

SUMMARY

From the experimental results of this investigation and a qualitative evaluation of the scale effect on the model study, the mechanics of sediment motion in the vicinity of a littoral barrier can be visualized as follows: The upcoast littoral current, and in some cases the shoreward motion along the bottom, bring the sediment toward the littoral barrier. In the neighborhood of the barrier, the high local turbulence created by and diffused from the barrier keeps the sediment in suspension. The suspended sediment is finally carried downcoast by the current along the barrier, due to the continuity of the flow. This description is based upon the deduction that in nature the turbulence at the front of the barrier is very much stronger than what can be reproduced in the laboratory scale.

ACKNOWLEDGMENT

This work represents research carried out for the Beach Erosion Board under contract with the University of California. The study was conducted under the guidance of Professor H. A. Einstein. For his most valuable advice and criticism, the writer wishes to express his appreciation. Grateful acknowledgment is made to Shang-Wen Yuan, graduate student at the University of California for his capable assistance in the experimental phase of this study.

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AN ELECTRONIC GAGE FOR MEASUREMENT OF SMALL WAVES AND RIPPLES

by
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Research Division - Beach Erosion Board

In laboratory wave action studies there occasionally are times when the wave heights become so low that with commonly used types of wave measuring equipment such as the step-resistance gage, the parallel-wire gage and the capacity gage, substantial errors in the recorded data are caused by the meniscus resulting from surface tension. The only way that the meniscus effect can be eliminated completely is to have some type of measuring device that does not touch the water surface. Such a gage has been developed, and is discussed below. The action of the gage depends on variation of the dielectrical capacity in the air space between the water surface and a probe, placed a short distance above, with changes in water level. This capacity is part of the tuning circuit of an oscillator. A variation of the water level changes the frequency of the oscillator. The output of this oscillator is mixed (heterodyned) with a constant frequency oscillator. The resulting beat frequency is fed into a tuned amplifier which is tuned so that any change of the beat frequency causes a rapid change in amplification.

A variation of the water-probe capacity caused by a change in the water level then results in a change in the beat frequency and a change in output of the amplifier.

The output of the amplifier is rectified and used to drive a Brush amplifier and recorder. The output can be made practically linear over a 0.2-foot water level variation, if the components of the circuits are properly adjusted.

Referring to the block diagram (Figure 1), the sections designated fixed oscillator and variable oscillator are two radio frequency generators. The fixed oscillator generates an alternating current of approximately 8.50 megacycles per second, and the variable oscillator approximately 8.93 megacycles per second, the difference being about 430 kilocycles per second, or the beat frequency.

The variable oscillator frequency can be made to vary by changing its capacity to ground. Referring to the variable oscillator (Figure 2) the dashed portion of the drawing, marked A in that section, represents the changing capacity when the gage is in operation. This changing capacity is formed by the probe acting as one plate and the water surface as the other plate of a condenser. When the water rises and falls the capacity changes due to the changes in spacing between the acting plates of the condenser.

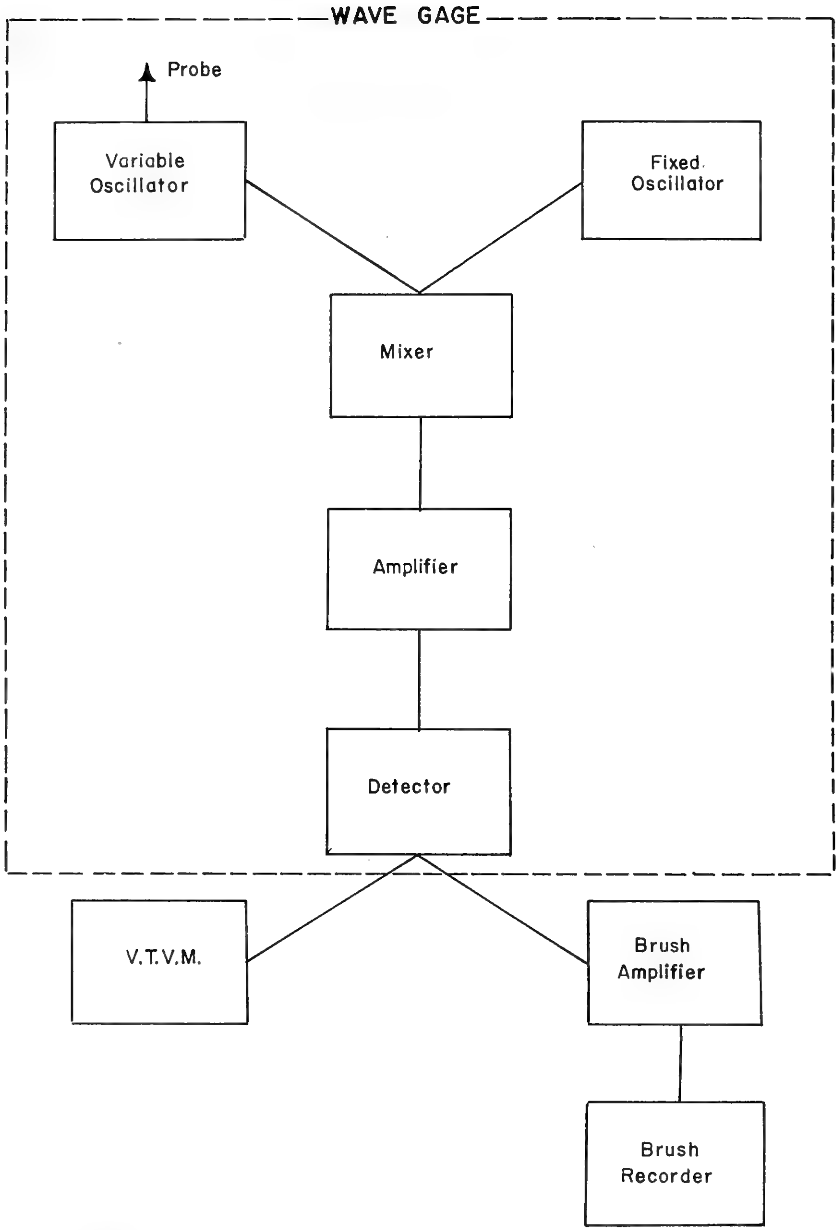

FIGURE I. BLOCK DIAGRAM FOR ELECTRONIC WAVE GAGE AND RECORDING EQUIPMENT

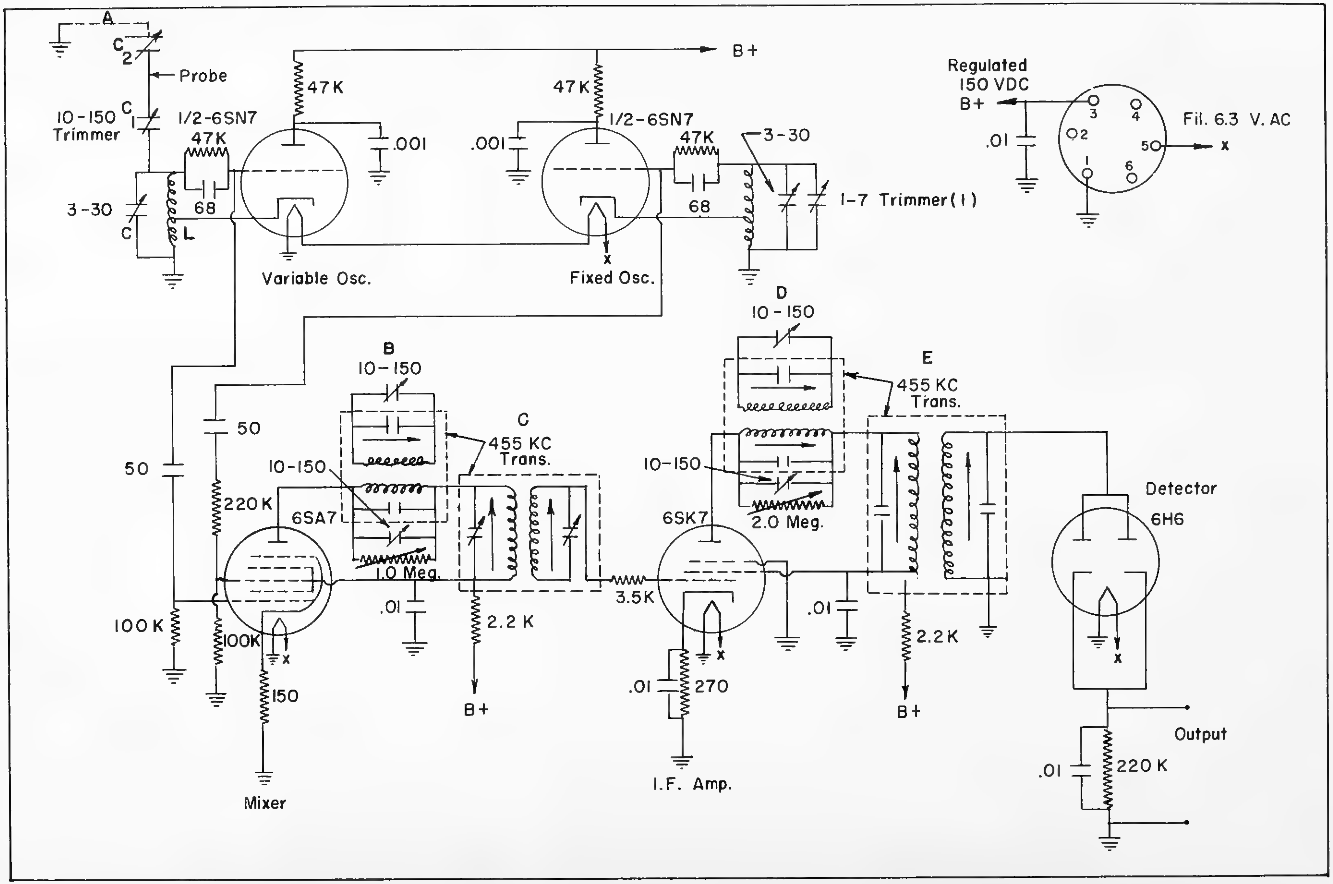

FIGURE 2. WIRING DIAGRAM FOR ELECTRONIC WAVE GAGE

The above variable frequency is mixed with the Radio Frequency (R.F.) voltage from the fixed oscillator. The purpose of mixing these two frequencies is to form a beat or intermediate frequency (I.F.). Since, the I.F. is the difference between the two R.F., the I.F. will have a greater change in proportion to its resting frequency in number of cycles per second than the R.F. from the variable oscillator, making the gage more sensitive to frequency changes.

Some of the power of the variable oscillator will be absorbed by the water and will vary with the distance of the probe from the water causing the output to the mixer to vary in amplitude, as well as in frequency. In order to prevent the effect of amplitude change from showing in the output of the gage, the voltage from the fixed oscillator is made smaller than the lowest value of voltage from the variable oscillator. When beating two frequencies together the resulting amplitude of the I.F. frequency can be no larger than the smaller of the two frequencies. One method of lowering the signal strength of the fixed oscillator is to drop it with a resistor network; in the circuit diagram (Figure 2) following the R.F. voltage path from pin 4 of the 6SN7, it will be found that the voltage from the fixed oscillator is divided by the two resistors 220K and 100K and the voltage is picked off from between the resistors, thus lowering the signal strength on the grid of the mixer (6SA7).

After the signal leaves the mixer tube it must pass two 455-kilocycle I.F. transformers. The one in Figure 2 marked B is used as a wave trap to shape the response curve of the output. The one-megohm potentiometer is used to vary the effects of the trap. The secondary of the B transformer will absorb some of the power in the circuit and further shape the output to make the gage linear. The transformer marked C is used to transfer the signal to the 6SK7 I.F. amplifier. The transformer C can also be tuned to make the output from the gage more linear. In the following I.F. stage, consisting of transformers D and E, the same action takes place, further amplifying and shaping the gage output for linearity.

The 6H6 detector tube is coupled to the output of the I.F. amplifier by transformer E. The signal is rectified by the diode of 6H6, changing the alternating voltage to a direct voltage which fluctuates as the distance between probe and water changes.

The output of the gage is fed into a Brush amplifier in order to match impedance with the Brush recorder. A vacuum tube volt meter is also placed permanently across the output of the gage so that the voltage output of the gage can be kept at the level yielding best linearity.

Relationships affecting the linearity of the gage response are changes in R.F. frequency of the variable oscillator due to changes

in distance from the water surface, and variation in gage output due to changes in the I. F. frequency. Neither of these relationships are linear, but can be shaped to make the output of the gage relative to its distance from the water surface a linear relationship.

The relationship of I. F. frequency vs. the output of the gage is a curve determined by the frequency response of the gage circuits from the output circuit of the mixer to the detector stage. If this curve can be shaped so that it will compensate for the curvature of the R. F. frequency vs. the distance from the water relationship, the output curve of the gage, as voltage vs. the distance of probe from water, will be a straight line. In Figure 3 a family of the latter curves is shown, in which curve No. 10 is almost a straight line. Each curve shown in Figure 3 represents use of a different frequency from the fixed oscillator. This frequency change of the fixed oscillator changes the voltage output of the gage for a fixed distance above the water surface. This frequency can be regulated by adjusting the small trimmer (marked (1) in Figure 2) in the fixed R. F. oscillator.

The performance of the gage as presently constructed is satisfactory, however, it is believed that some improvements can be made. The instrument drifts with temperature changes as it warms up but if allowed to warm up for 45 minutes it is very nearly stable. When used in a building where temperature does not change rapidly the gage can be used very satisfactorily but will have to be adjusted occasionally. Calibration drift due to temperature change is shown in Figure 4 to be about 0.067 volt per degree centigrade of temperature change over a range of 0.19 foot. Tuning and adjustment of the gage for linearity must be carefully done; the adjustment that effects linearity the most seems to be in the probe coupling condenser and a larger trimmer in this circuit might help.

The power supply has a voltage regulating (V.R. 150) tube across the B+ and a constant voltage transformer input to eliminate effects of line voltage changes.

The probe used in this gage was made from a brass rod 1/16 inch in diameter and 5 inches long, with a circular plate 3/16 inch in diameter soldered to the end next to the water surface and the other end bent in a loop for fastening to a ceramic feed-through. Other sizes of probes could be used but the gage circuits would have to be readjusted.

To adjust the gage for linearity a family of curves as shown in Figure 3 must first be made. This family of curves is obtained by adjusting the Probe coupling, I.F. transformers, and the trimmer in the fixed R. F. oscillator tank circuit. By varying the tuning and coupling condensers a curve approaching a linear relationship, such as No. 10 on Figure 3, can be found. This curve then is the

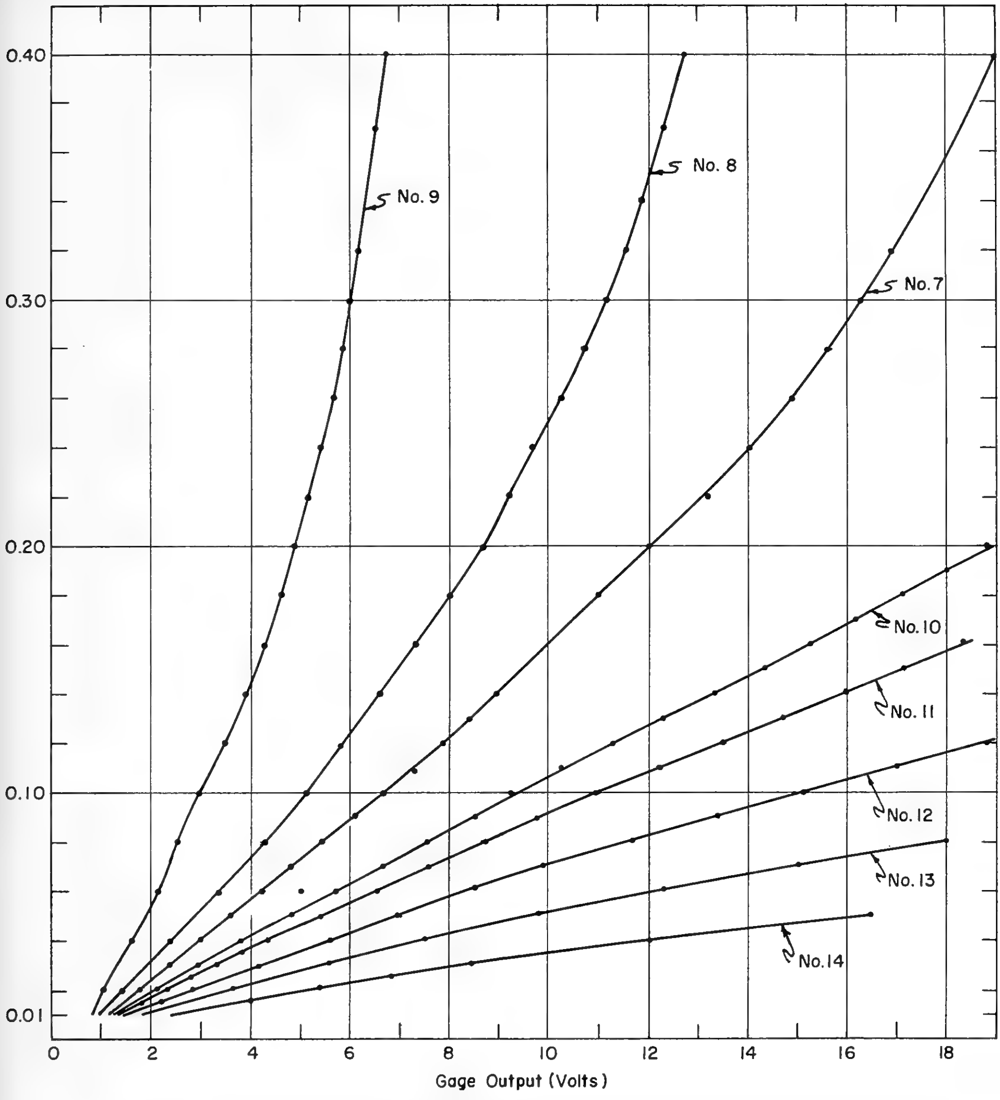

FIGURE 3 · GAGE OUTPUT (VOLTS) VS PROBE DISTANCE FROM WATER SURFACE FOR DIFFERENT FREQUENCIES OF FIXED OSCILLATOR

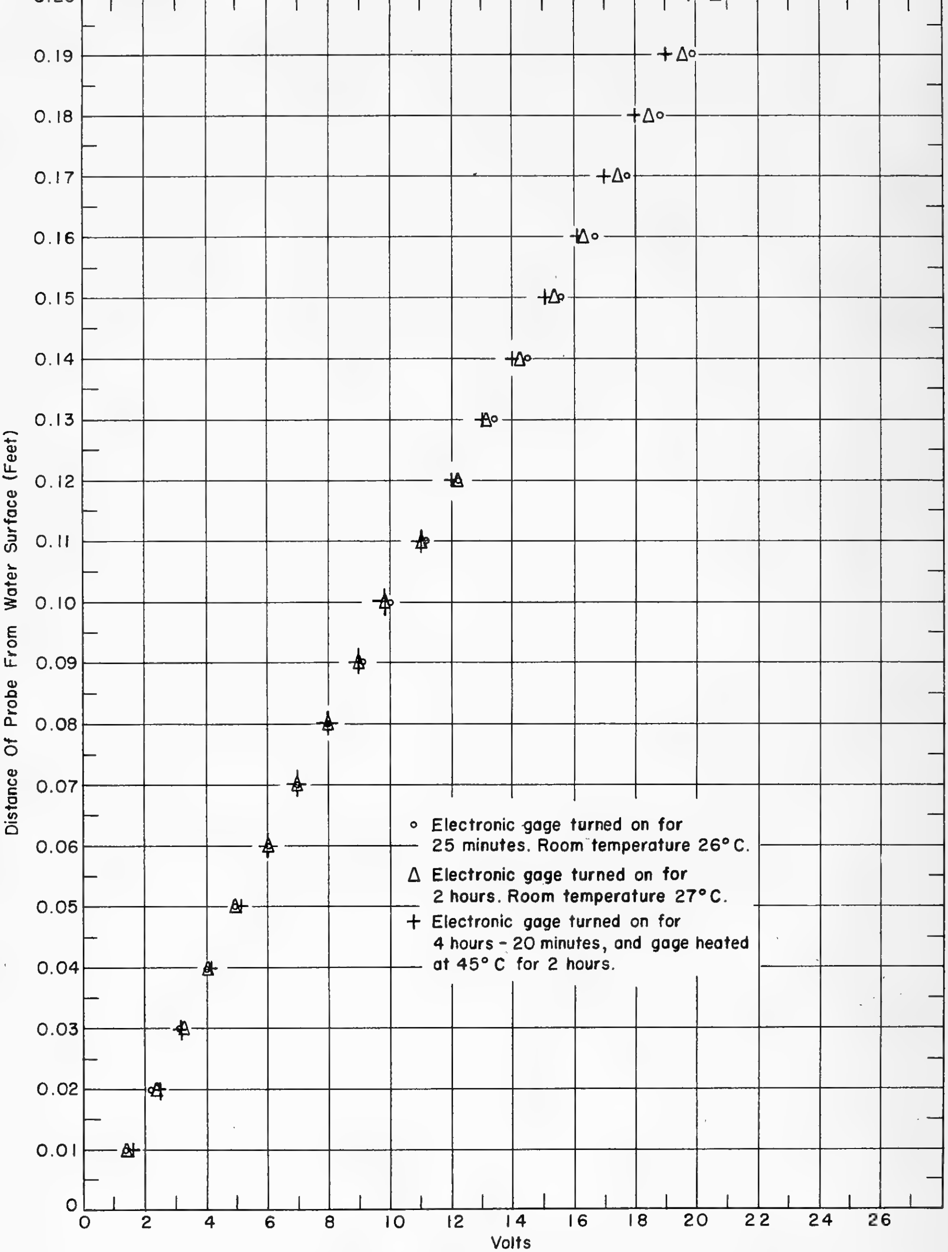

FIGURE 4 · CALIBRATION DRIFT DUE TO TEMPERATURE CHANGE

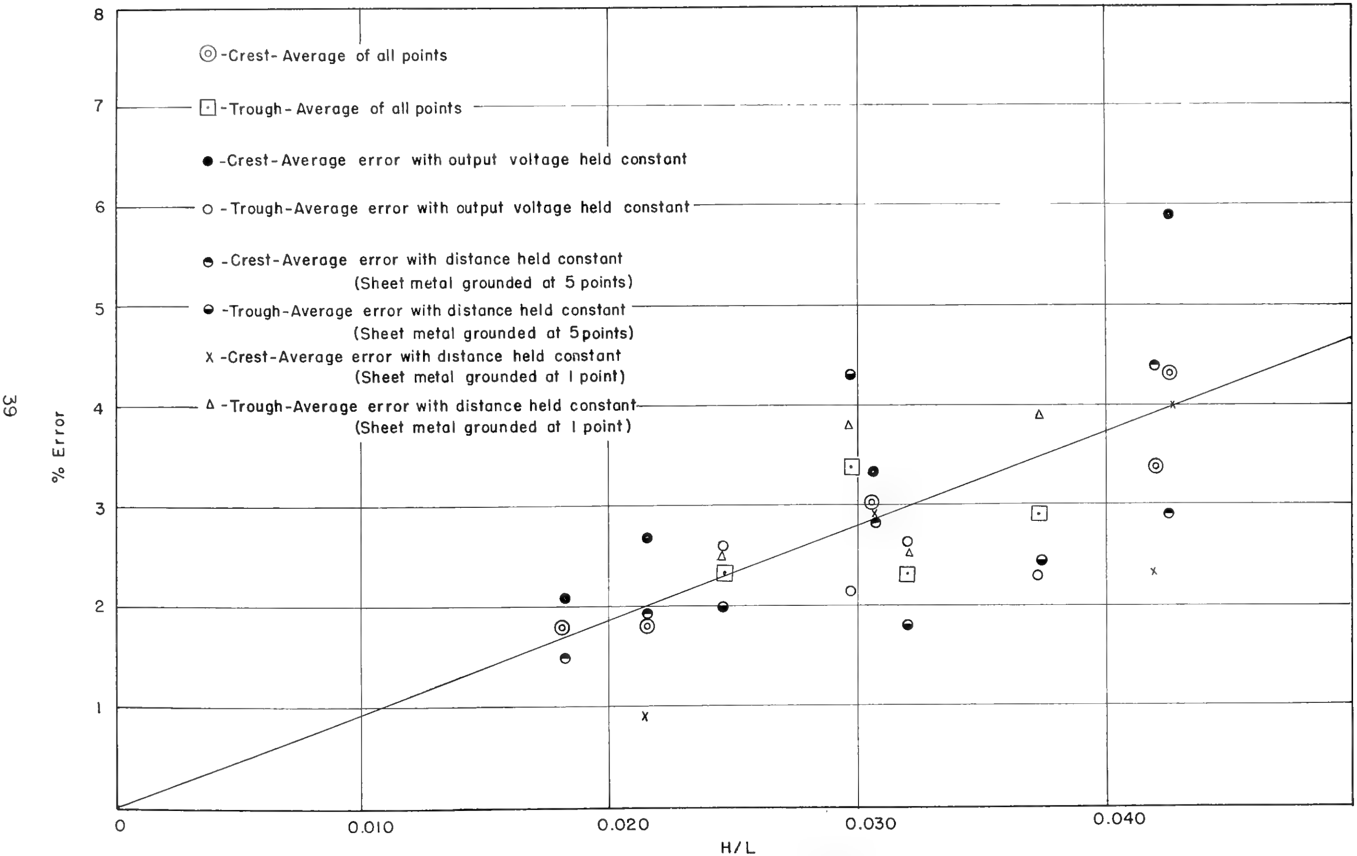

FIGURE 5. PROBABLE ERROR IN GAGE PERFORMANCE DUE TO WAVE SHAPE

curve for linear calibration. To operate the gage after the gage has been made linear, the only adjustment necessary is that with the trimmer in the tank circuit of the fixed R. F. oscillator.

Figure 5 shows the error that can be expected due to wave shape. Since this gage was thought to be more accurate in the measurement of small waves than any other means available, sheet metal shaped in the form of a train of water waves was used to find the probable error due to wave shape. The shaped sheet metal was put on a carriage which ran on a track under the gage. The gage was made adjustable as to its height above the shaped sheet metal. The formed sheet metal waves ranged from 1/4 to 1 inch in height and 1 to 5 feet in length. At one end of the carriage there was a flat plate over which the gage was calibrated; it was then assumed that the calibration would be the same over still water. After calibration, and in order to establish the effect of surface curvature on the calibration, the carriage was moved under the gage until either a crest or trough was directly under the gage. Without any changes or adjustment of the gage circuit, the gage elevation was reset to maintain a constant distance above the formed sheet metal and a voltage reading was then taken. Following this test, with no change in gage circuit and with the same conditions as above, the gage position was reset to maintain a constant voltage output from the gage, then a reading of the gage distance above the formed sheet metal taken. The percent of change corresponding to the reading over the flat plate was plotted against ratio of the height of the formed wave to the wave length and is shown in Figure 5; the error indicated on this plot representing the errors introduced by the curvature of the surface of the wave train.

DEVELOPMENT OF A SUSPENDED SEDIMENT SAMPLER FOR LABORATORY USE UNDER WAVE ACTION

by

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INTRODUCTION

General. Studies of sand (littoral) movement in the recently completed Shore Processes Test Basin of the Beach Erosion Board will eventually require the quantitative measurement of the amount of sand thrown in suspension by wave action. On an open coast, this sand would be available as suspended load for transport by the longshore currents developed by the wave action. Pump-type samplers have heretofore been used successfully in the field under conditions of oscillatory flow and to some extent in the laboratory. It was felt that this type of sampler probably offered the best chance of successful operation, and certain exploratory tests were carried out to determine the best working procedures, and to establish some idea of the degree of error and reproducibility to be expected.

The accuracy and efficiency of numerous types of samplers for use in unidirectional stream flow have received extensive study and investigation(1)* and, in general, their use in river flow is considered quite satisfactory. For efficient operation however the intake nozzle should be oriented directly into the stream flow. Such orientation in oscillatory flow is, of course, impossible due to the reversible flow accompanying the wave action which causes the sand to be intermittently in motion first in one direction and then in the other.

Review of the literature treating sediment suspension under wave action revealed no direct method whereby the true suspension at a point can be determined; however, a report by Watts(2) presents the results of the laboratory development and field testing of a pump-type suspended sediment sampler for use in (full scale, ocean) wave action. The report gives a sampling procedure designed to collect a suspended sample representing the average concentration over a number of wave cycles.

Due to the absence of any reliable method applicable to wave action for determining the instantaneous concentrations at a point and the expected complexity plus added cost of considering an instantaneous sampler, it was decided to collect the fluid-sediment mixture over some measured period of time as had been done in the field sampling. This would give a concentration value averaged over a number of wave cycles. It was felt that such long-time samples would minimize the variance between successive values and therefore more accurately reflect the average concentration. The greater variance for a short interval sample results from the erratic nature of the turbulent forces influencing the concentration; by sampling over several wave cycles, however, the effects

*Numbers in parentheses refer to references at end of report.

of these forces are averaged out, and a concentration value more representative of the average wave conditions measured is obtained.

In general, the suspended sediment concentration resulting from a given wave condition varies with distance from shore, usually being a maximum in the zone of wave breaking. In developing the sampler, it was judged expedient to limit the overall length of the sampling area, thus obtaining somewhat limited total variability of sediment concentration in order to have the data more sharply focused on sampling procedure. The samples were actually collected in an area one to two wave lengths seaward of the breaker zone. The breaker zone was not included.

Purpose. The primary purpose of the study was to investigate by exploratory sampling in wave action those factors of sampling equipment, procedure, and environment (local bottom irregularities) tending to influence the accuracy of suspended sampling data.

In particular it was desired to study the effect of pumping (or sucking) sediment laden water through an intake nozzle positioned at some point above the sand bed. Visual observations of this operation combined with repetitive samplings and analysis would provide the basic sampling design criteria. This basic design criteria could then be systematically improved by studying the inter-relations of nozzle disturbance to flow, fluid-sediment flow, and local bottom irregularities. All these factors in greater or less degree affect the exactness of agreement between a pumped sample and the actual suspension at the sampling point. This type of study would provide the necessary data to evaluate the test results in terms of the degree of sampling reproducibility. Sound recommendations could then be made for improving the reliability of the sampling procedure.

Definite criteria for nozzle size, shape, orientation, filtration technique, etc., were not decided before the testing began; but rather the project actually began by collecting samples using a relatively arbitrary setup of sampling apparatus and proceeded to study, from test observations and analysis, the changes in test setup or procedure which improved the accuracy of the sampling operation. The ultimate objective was then to select from the numerous sampling setups the particular procedure found to produce the most satisfactory results.

EQUIPMENT AND PROCEDURE

General Testing and Apparatus. The tests were made in a concrete wave tank 85 feet long, 14 feet wide and 4 feet deep (Figure 1). The tank had been partitioned lengthwise into four sections for previous tests, each of the separate sections containing sediment with a different distribution of grain sizes; the median grain diameters being 0.22 mm., 0.47 mm., 1.20mm., and 3.44 mm. respectively. Suspended sediment sampling

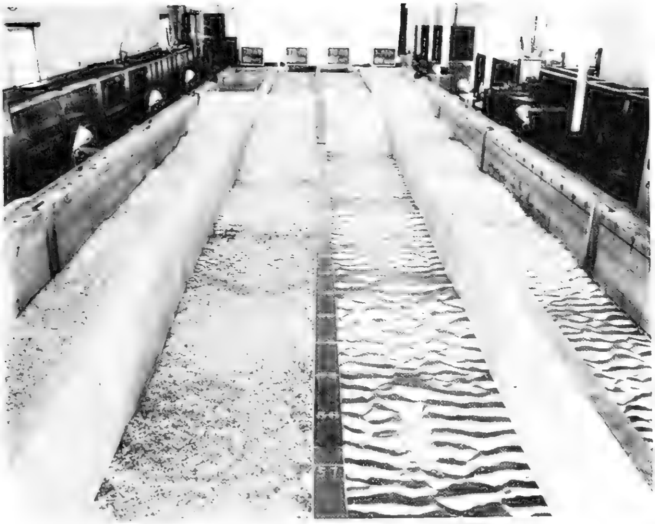


FIGURE 1. WAVE TANK

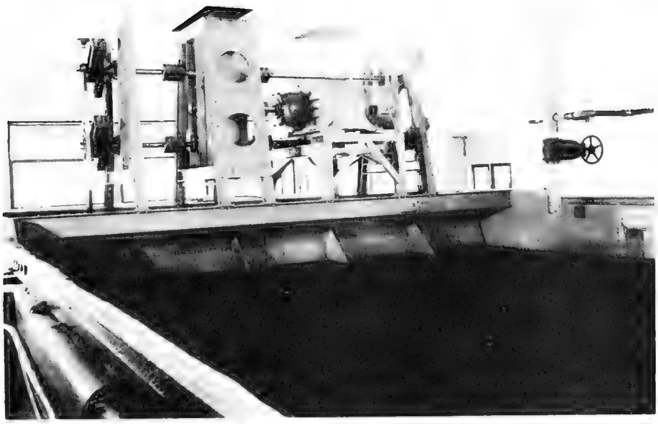


FIGURE 2. WAVE GENERATOR

however, was done only in the two sections of the tank containing the finer sized sediments. Figure 3 shows the size distribution curves of the 0.22-mm. and 0.47-mm. sediments. The wave tank was provided with a bridge-type carriage spanning the tank width and equipped with rollers for convenient movement to any desired sampling location along the tank. The entire sampling apparatus was mounted on the tank carriage making it possible to conveniently position the intake nozzle at any desired sampling location. Waves were produced by a curved-face bulkhead type generator moving through a forward and backward rolling motion made possible by the use of an eccentric drive (Figure 2). Wave heights and thus wave steepness ratios could be varied by adjustment of the eccentric setting. The generator was powered by a vari-drive motor permitting a wave period range of 0.9 to 3.5 seconds.

Depth and Sampling Elevation Measurements. It was desired to keep the depth as nearly constant as possible during the actual pumping operation; hence prior to sampling an equilibrium profile was allowed to form for the wave conditions tested. Despite this, however, due to the wave action the bottom configuration was constantly changing with time, primarily due to ripple formation and movement. This bottom ripple movement resulted in a changing depth at any point which, though slight, was still significant. It was thus necessary to determine an average depth. taking into account the bottom changes as evidenced by the elevation change of a given ripple crest or trough during the pumping of a sample (Table 1, Column 1). The actual average depth for a given instant was then determined as $(h_R/2) + E$ where h_R was the ripple height (trough to crest) and E was the vertical distance from the ripple crest to still water level. The tabulated depths however, reflect the average of two values of the quantity, $(h_R/2) + E$, one taken before starting the sampling and a second one approximately five minutes later, after completion of sampling.

The necessary measurements to determine depth, sampling elevation above bottom and still water level were made using a vertically oriented point gage with a vernier scale mounted to the wave tank carriage (Figure 4). A similar type vernier scale was used to measure the height of the nozzle above bottom.

Wave Measurements. A high speed pen-and-ink Brush oscillograph and a parallel wire resistance wave gage were utilized for recording the pertinent wave data. A continuous wave record was obtained covering each sampling period. The wave recording equipment was calibrated before each series of sample collections and the calibration checked at the end of the sampling series. Average wave heights were determined from the wave record for each sampling, and the wave height thus determined for a particular sampling was the wave height used in summarizing the results. Figure 4 also illustrates the wave height measuring and recording equipment schematically.

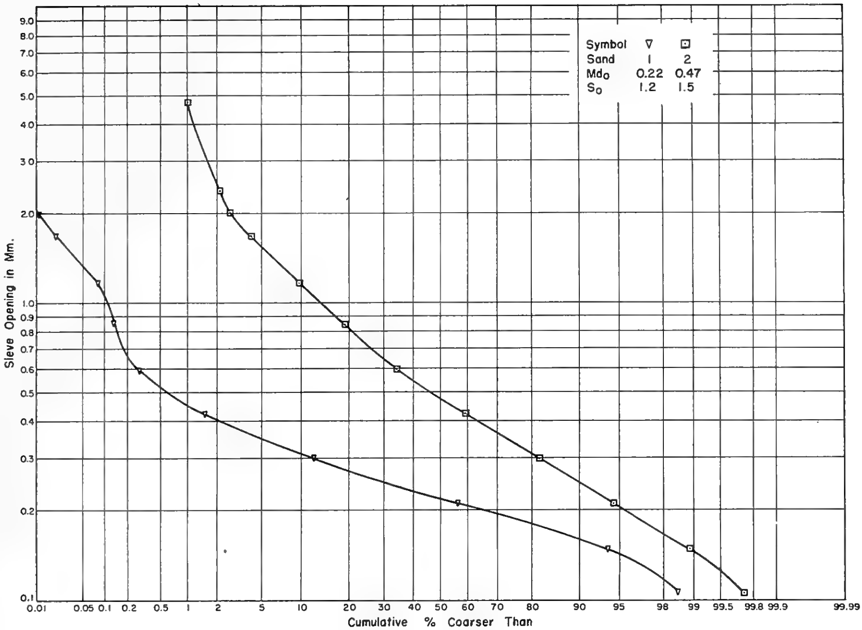


FIGURE 3 · SIZE FREQUENCY BY WEIGHT OF BED MATERIALS USED IN THE STUDY

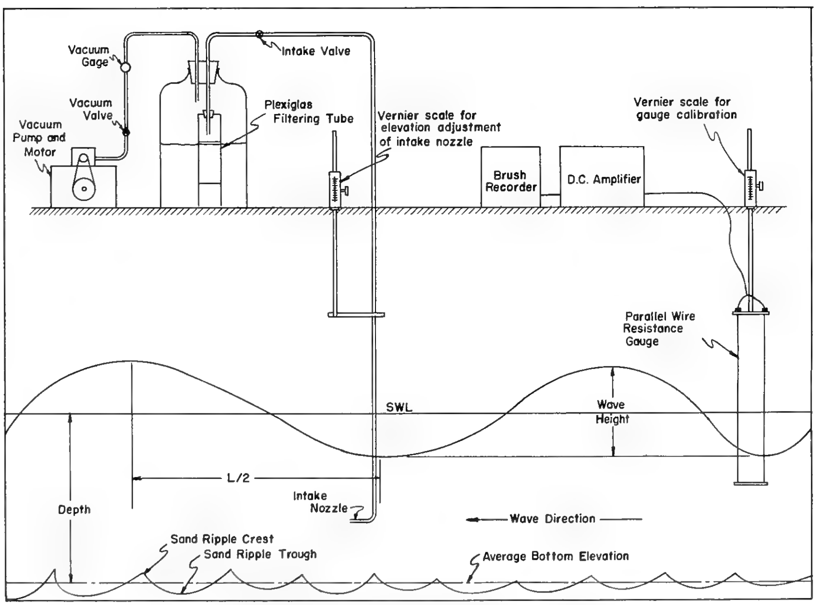


FIGURE 4 · SCHEMATIC DIAGRAM OF TEST SET-UP

TABLE 1

Elev. Change of Ripple Crest (foot)	Elev. of Nozzle Above Bottom (foot)	Wave Height (foot)	Sampling Volume Milli-liters	Sampling Time Seconds	Sediment Weight Grams	Concentration grams/cm^3 (x 10^{-3})
PART A - Nozzle Oriented Directly Over Sand Ripple Crest						
0	.105	.67	7600	93	14.45	1.91
-.025	.167	.62	7575	96	10.45	1.38
+.010	.195	.66	7500	95	2.25	0.304
-.010	.305	.66	7550	95	0.18	0.024
-.015	.457	.67	7525	95	0.08	0.0105
+.010	.095	.68	7525	112	20.20	2.96
-.030	.165	.68	7550	98	3.37	0.45
+.005	.198	.67	7550	95	1.61	0.214
-.020	.310	.69	7600	98	0.15	0.0196
-.005	.453	.67	7500	98	0.15	0.0196
0	.10	.62	7600	96	25.65	3.39
-.010	.155	.62	7575	96	6.89	0.912
-.010	.205	.62	7575	97	3.59	0.475
-.010	.305	.62	7600	95	0.70	0.0921
-.005	.452	.60	7625	96	0.29	0.0384
+.020	.090	.58	7600	93	6.58	0.865
-.030	.165	.57	7900	96	1.28	0.163
+.016	.192	.58	7600	94	1.91	0.254
+.014	.293	.58	7650	93	0.84	0.112
0	.450	.64	7600	98	0.39	0.051
0	.10	.62	7600	100	28.08	3.70
+.005	.148	.60	7600	99	10.25	1.35
-.025	.212	.62	7550	99	1.40	0.188
-.010	.305	.62	7600	97	0.61	0.081
0	.450	.62	7500	100	0.26	0.035
PART B - Nozzle Oriented Directly Over Sand Ripple Trough						
+.020	.09	.68	7600	96	9.93	1.31
-.035	.158	.64	7525	94	2.32	0.31
-.015	.207	.69	7600	95	1.20	0.162
+.015	.282	.64	7575	95	0.40	0.053
0	.450	.66	7575	96	0.16	0.021
-.015	.107	.55	7550	96	2.00	0.276
-.015	.158	.53	7575	95	0.60	0.079
-.010	.205	.53	7600	96	0.52	0.068
+.030	.285	.53	7550	96	0.31	0.041
+.010	.445	.51	7550	95	0.12	0.0161
0	.10	.57	7575	100	8.28	1.09
+.010	.145	.57	7700	104	4.53	0.59
+.006	.197	.57	7600	103	2.81	0.37
-.010	.305	.59	7575	104	1.57	0.21
+.010	.447	.58	7575	97	0.50	0.066

Sampling

Pumping System and Apparatus. The sampling was done by pumping (or sucking) the sediment-laden water through an intake nozzle into a collecting and settling tank (Figure 5). The pumping was accomplished by a vacuum pump (Figure 6) driven by a 1/4 horsepower motor capable of maintaining a vacuum inside the collecting tank equivalent to 26.5 inches of mercury. The collecting tank (Figure 5) was a 2-½ gallon pyrex bottle equipped with a two-hole rubber stopper, which permitted air-tight entry of two pipes. The two pipes entering the collecting tank were made of copper tubing; one served to connect the vacuum pump to the collecting tank; the other served as an intake line for the sediment-laden water. The connecting pipes for the entire pumping system were made from copper tubing with sections of rubber tubing inserted where necessary to obtain flexibility. The pump, piping, valves, and collecting tank comprised a closed system necessitating the use of air-tight valves and connections throughout.

It was desired to keep the intake nozzle and that portion of the tubing inserted in the water as small as possible to minimize effects on the flow regime, but yet it had to be large enough to permit easy flow of the fluid-sediment mixture through the line without clogging. Quarter-inch copper tubing was finally selected as the best compromise between these two requirements and was used in all tests.

Filtration Devices and Methods. Several mechanisms were tested to extract the sediment from the fluid-sediment mixture. The first, and most obvious, was simply to use the two-gallon pyrex bottle as a settling container with subsequent decantation of the excess water. The remaining water was then boiled from the wet sediment mixture leaving only the dry sample. This method was cumbersome and time consuming and was therefore discarded; yet it served a useful purpose in that it gave a rough estimate of the range of sediment concentrations to be expected in the sampling tests.

A second device for extracting the sediment from the fluid-sediment mixture consisted of a clamping mechanism housing a filter and equipped to fit water tight over the mouth of the pyrex collecting tank (Figure 7a). It was of plywood construction and required that the collecting tank be inverted to allow the water to drain through filter paper and thus separate the sediment from the water. A pressure equalizing tube (Figure 7b) was necessary to maintain atmospheric pressure above the water surface inside the inverted bottle thus aiding the effective force on the filter and therefore the filtering rate. This method of sediment separation was accurate and worked fairly well but lacked the flexibility of the method which follows.

The third system of sediment separation employed the use of filters throughout the collection and separation process. The fluid-sediment

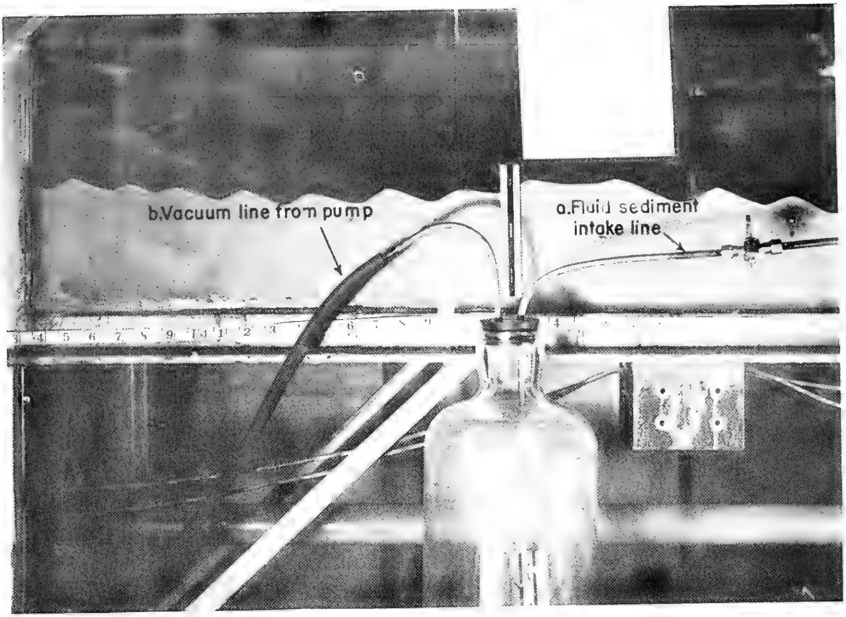


FIGURE 5. SEDIMENT COLLECTION TANK, SHOWING INTAKE AND VACUUM LINES.

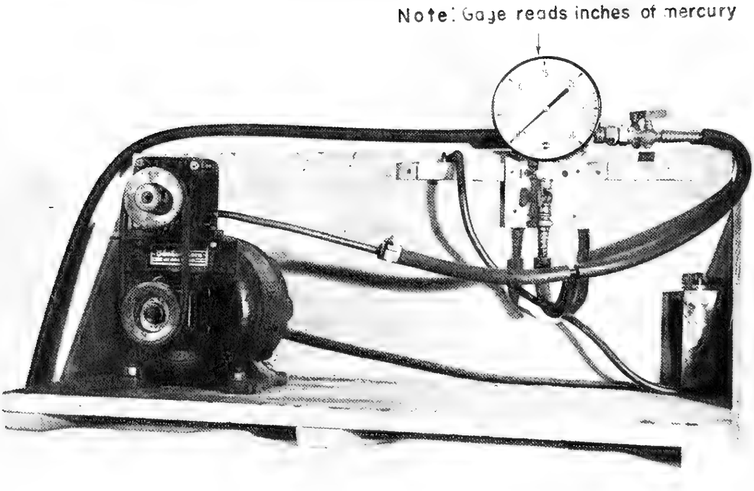
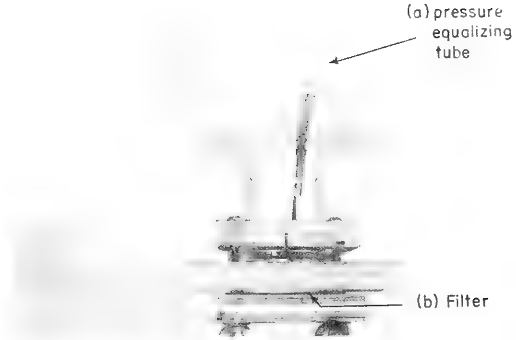
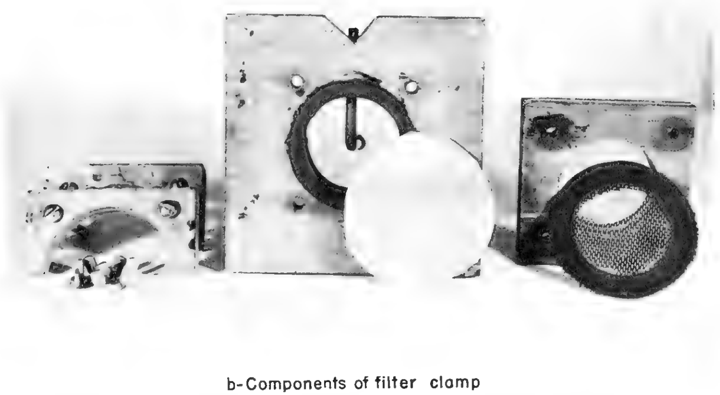


FIGURE 6. VACUUM PUMP AND GAGE



a- Collecting tank with filter attached



b-Components of filter clamp

FIGURE 7. FILTER CLAMP ASSEMBLY FOR COLLECTING TANK

mixture from the sampling point was pumped through the intake line into a filtering tube placed inside the collecting tank (Figure 5). The filtering tube (Figure 8) was rectangular in shape, was made from plexiglas, and was of water-tight construction; panel sections of the rectangular tube were left open and covered by filters using water-tight seals to attach the filters to the tube. A rubber stoppered hole at the top of the filtering tube permitted entry of the 1/4-inch intake line. A small removable panel at the base of the filtering tube facilitated convenient extraction of the sediment sample. The collection and filtration process employing the use of the plexiglas filtering tube was considered the most satisfactory method and is currently in use at the Beach Erosion Board laboratory. The filtering tube as designed in the test program and illustrated in Figure 8 is simple and economical to construct. Its use simplifies sample collection and facilitates subsequent extraction of sediment which should minimize some of the inherent error of suspended sediment analysis. The sampling tube can further be adapted to several types of filters and thus more readily fit the needs of the user. The entire sampling apparatus is portable and may be quickly set-up for the collection of suspended sediment samples.

Collection of Fluid-Sediment Mixture. The actual operating procedure for collecting the desired volume of fluid-sediment mixture for subsequent extraction of sediment by any of the above filtration methods was as follows: (1) with the intake valve closed, the vacuum pump motor was started and run for sufficient time to reach a maximum and constant gage reading. This gage reading on a 3-inch vacuum gage remained (fairly) steady at 26.5 inches of mercury during the pumping of a sample. (2) While maintaining a constant gage reading, the intake valve was opened for sufficient time to collect the desired volume of the fluid-sediment mixture; this volume was measured in milliliters by a graduate scale attached along the side of the pyrex collecting tank. The sampling time (actual pumping time) needed to collect the volume of fluid-sediment mixture was measured by stop-watch.

Sediment Drying and Weighing. After collection of the sample in the filtering tube, it was removed by washing onto ordinary filter paper (Figure 9), air dried at room termperature and then weighed on laboratory scales. The weight of the dry filter was recorded prior to its use in the collection process and later subtracted from the gross weight of dry sediment and filter in order to determine the net weight of dry sample. The weight in grams thus determined was tabulated for use in summarizing the test results.

Orientation of Intake Nozzle

A factor of primary importance to the sampling operation is the nozzle orientation and its degree of disturbance to the flow regime. The particular nozzle orientation evidencing the least disturbance to the flow regime is obviously the one most likely to insure collection of a

FIGURE 8. FILTERING TUBE

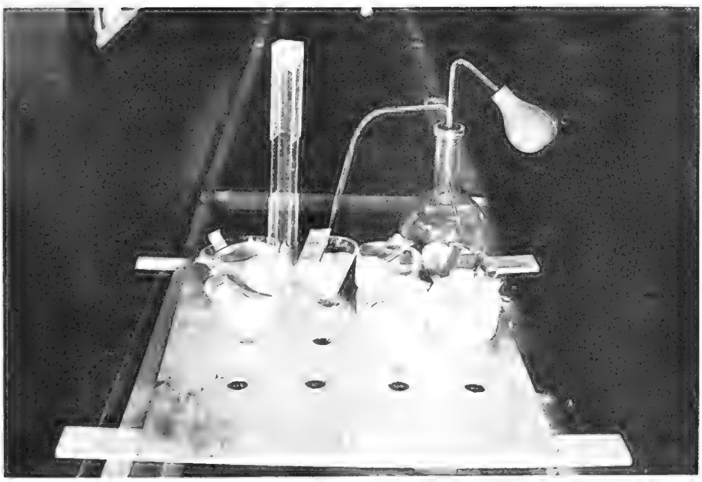


FIGURE 9. SAMPLE AND FILTERS REMOVED FROM FILTERING TUBE FOR DRYING

sample most nearly representative of the true suspension. Consequently certain preliminary samples were collected using different nozzle orientations (Figure 10) in order to determine the most suitable orientation. It was believed that the inherent nature of the forces accompanying the wave action and the resulting complexity of flow disturbance with any type of nozzle or orientation precluded any clear-cut selection of the best nozzle orientation (or shape). From close observations of the various criteria presumably influencing the accuracy of the several nozzle orientations and the analyses of preliminary sampling tests it was decided to use in the tests a nozzle intake orientation directed normal to the oscillatory (stream) flow.

ANALYSIS OF RESULTS

Sediment Concentration. The suspended sediment concentration is generally defined as the percent by weight of sediment in a given volume of fluid-sediment mixture, flowing through some specified cross-section of area. If we assume a uniform concentration for successive cross-sections along and perpendicular to the flow, then the average and the instantaneous concentrations would everywhere be equal. While this may occur under carefully controlled ideal conditions for unidirectional flow, it does not for pulsating or reversing flow -- as with waves. Hence any concentration figure represents a certain average value.

The sediment concentration by weight as reflected by an individual sample was determined from the net weight of the sample and the measured weight of the water and sediment collected. Figures 11 and 12 present experimental curves showing the relation of average suspension concentrations to such parameters as wave height, depth, velocity, etc.

The value of a sediment sampling process depends largely on its reproducibility under identical sampling conditions. The curves of Figures 11 and 12 showing the variability of sediment concentration versus elevation above bottom show relatively poor consistency for the same wave conditions. Care was exercised to maintain constant conditions as to wave parameters and pump-sampling procedure, but even with close controls, the cloudlike formations exhibited by the suspended particles varied considerably with time.

Sediment Concentration and Local Bottom Irregularities. Visual observations of the clouds of suspended particles with the interspersed voids or areas of minimum suspension provided some explanation for the wide scatter of the sampling data. The observations revealed that the clouds and voids (areas of minimum suspension) were constantly shifting about. The shifting was gradual and in two directions; one was along the direction of wave travel concurrent with the shoreward movement of the sand ripple; the other was along the length of the sand ripple and coincided rather closely with local changes of ripple height and curvature. The sand ripple movement along the direction of wave travel was estimated to range from 1 to $1\frac{1}{2}$ inches for the sampling period which averaged 100 seconds.

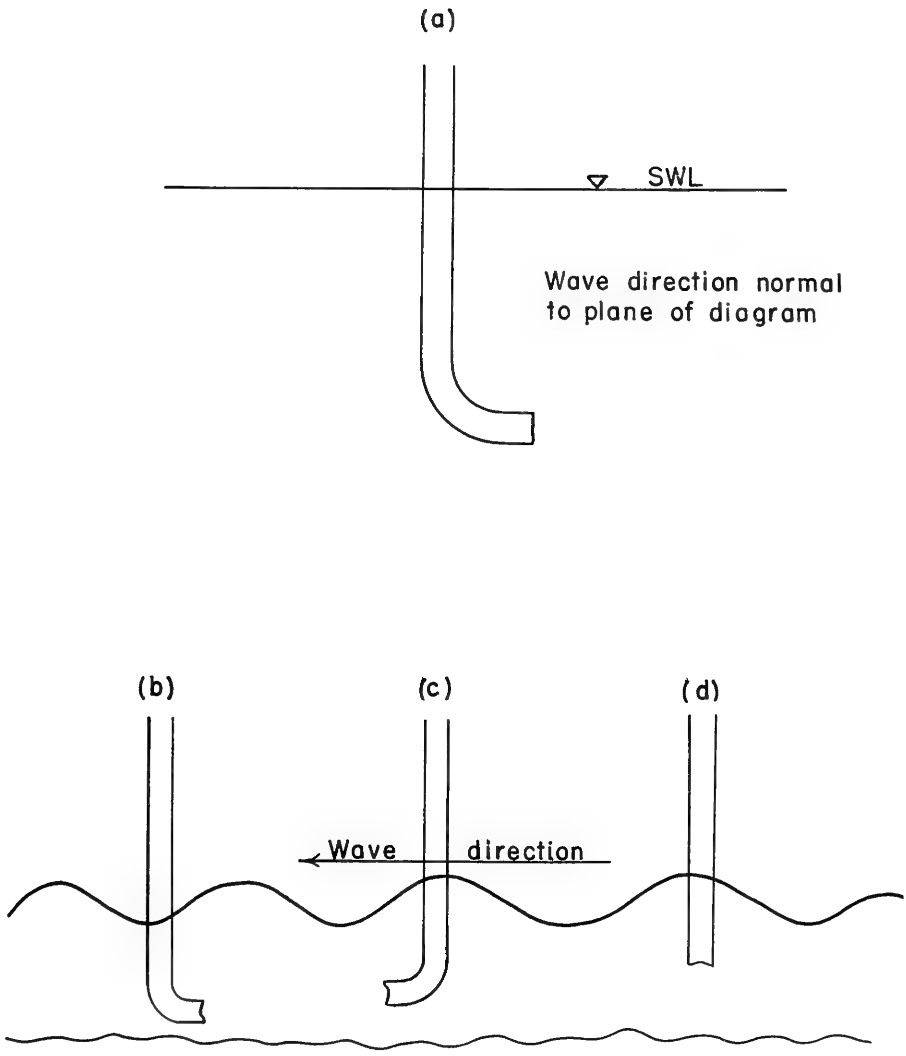


FIGURE 10. INTAKE NOZZLE ORIENTATIONS RELATIVE TO WAVE DIRECTION

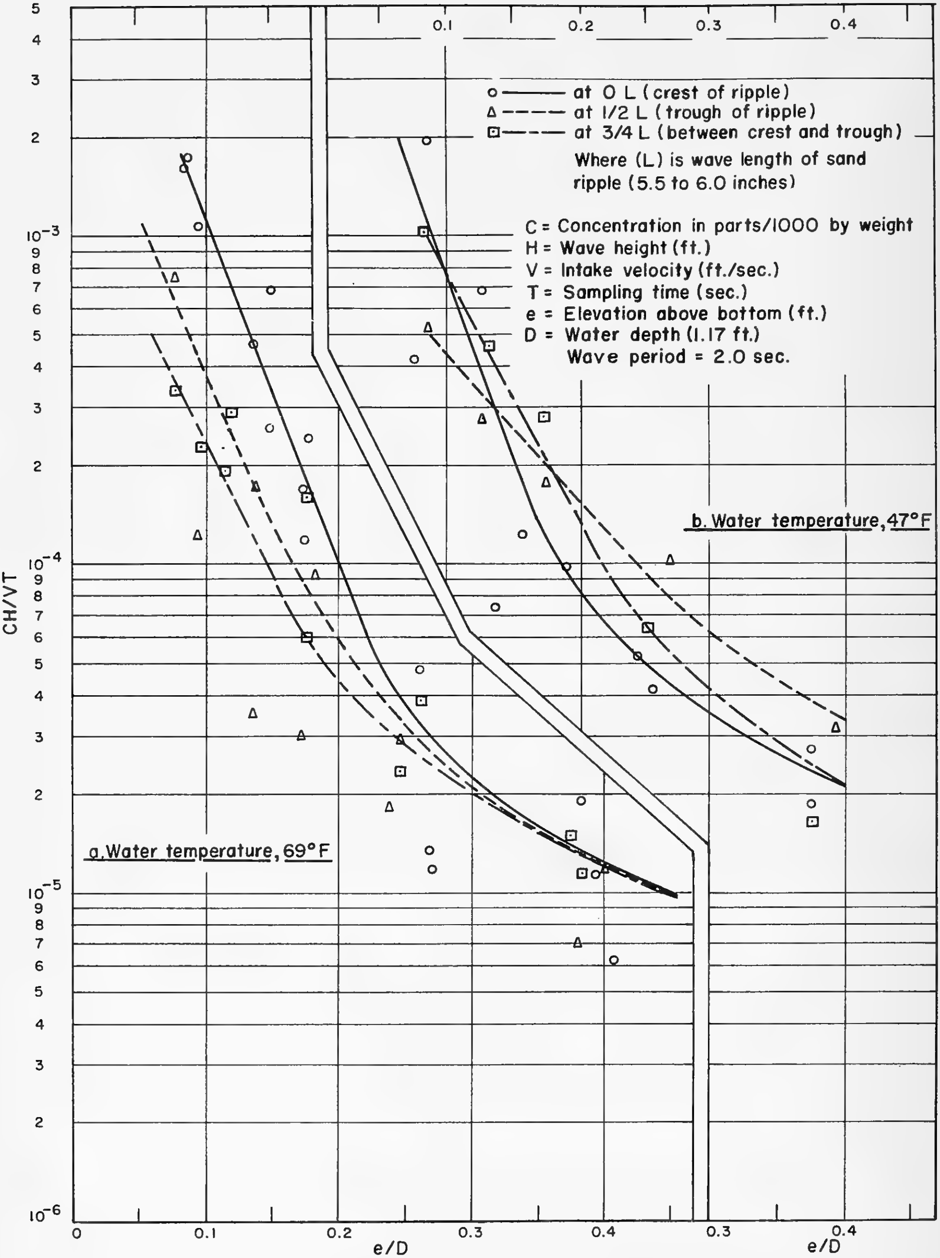


FIGURE II · SUSPENDED SEDIMENT CONCENTRATION AT THREE NOZZLE POSITIONS ALONG THE SAND RIPPLE PROFILE

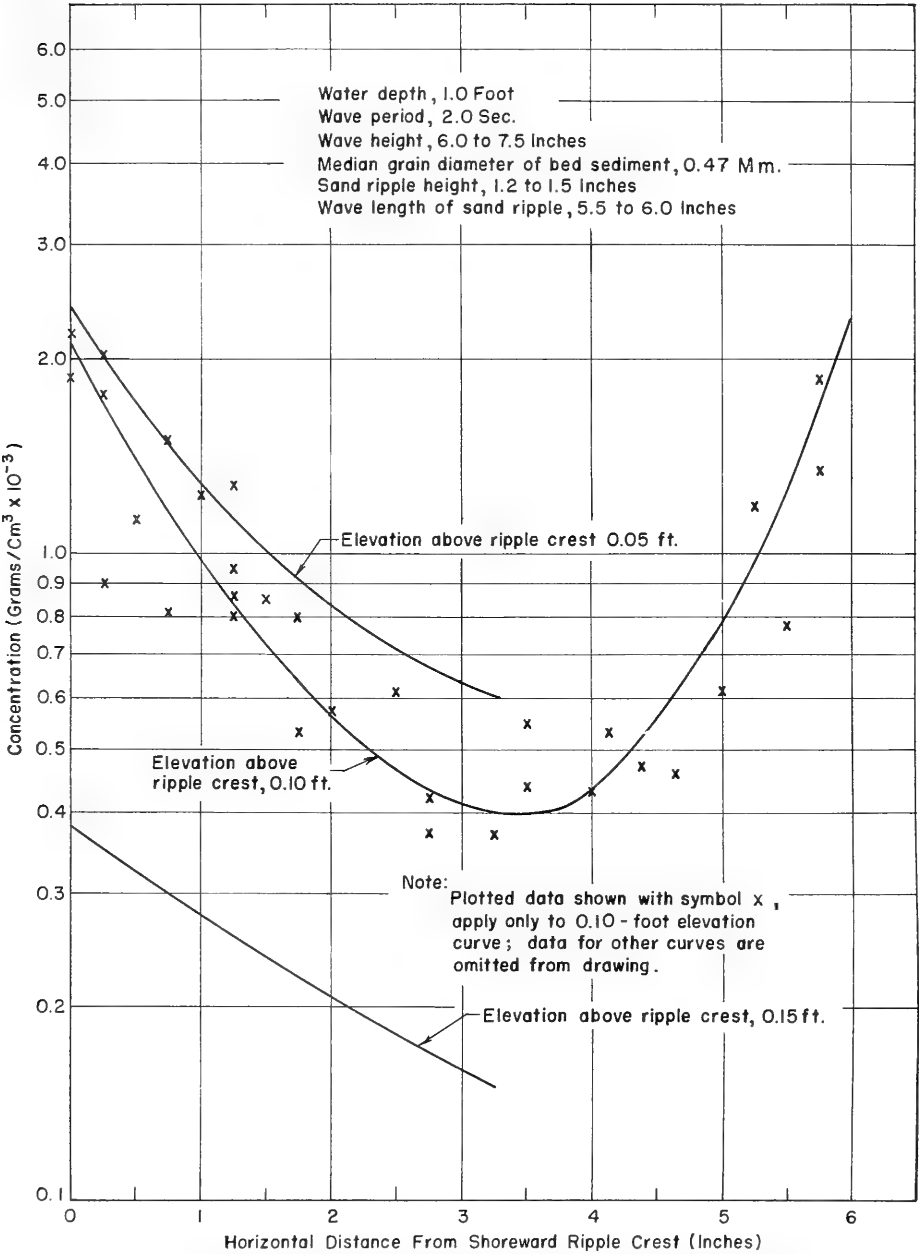


FIGURE 12 · SUSPENDED SEDIMENT CONCENTRATION VS HORIZONTAL DISTANCE FROM SAND RIPPLE CREST

In order to investigate this relation, measurements of the change in ripple height were made and tabulated along with depth, sediment concentration, wave height, etc. This data is also contained in Table 1. Column 1 of Table 1 gives the changes in ripple height which is seen to vary significantly, but in an apparently random fashion. The actual ripple height changed as much as 0.035 foot (or 35% of its maximum height of 0.1 foot) in roughly 100 seconds.

The constantly changing ripple height, represented essentially a constantly changing depth. This depth change imposed an interaction between the bottom and the turbulent fluid motion, which rendered improbable the existence of a uniform pattern of sediment concentration. Due to the random variation in magnitude and direction of the forces accompanying the orbital currents and turbulent motion associated with the wave action, the concentration pattern was very erratic and non-uniform; that is, it exhibited a wide range of total suspended load, which apparently followed a chance distribution.

The random spacing of the suspended particle clouds was visibly apparent when viewed along the sand ripple length. The clouds and voids of suspended particles were most sharply defined directly above the sand ripple crests and at the instant of wave crest passing. During the instant of flow reversal, near and along the bottom, the major portion of the suspended sediment load appears to be concentrated directly above the sand ripple crests. In contrast for the same instant of time there appears a scarcity of suspended particles directly above the sand ripple trough. For the same wave conditions tested the analysis and comparison of the data indicate the existence of much higher sediment concentrations directly above the sand ripple crest, as contrasted to the very low concentrations found directly above the sand ripple trough. As a consequence of this sharp sediment concentration gradient from sand ripple crest to trough the selection of the sampling nozzle position becomes important in laboratory measurements. Therefore, the nozzle position for laboratory sampling measurements should be carefully selected and thus become part of the sampling data and analysis.

The data on several samples for each of the three nozzle orientations (illustrated in Figure 10 a,b, and c) are contained in Table 2. A comparison of the expected consistency of sampling results based on the three sets of data was inconclusive. In the first place there was not enough data and secondly the wide scatter of the data reduces any conclusion to doubtful speculation. Despite this however, it was interesting to note the apparently higher values of sediment concentration and somewhat better consistency of results obtained, by using a nozzle oriented normal to wave crest and pointed seaward.

At this point the question may be asked as to why the tests were conducted using a nozzle orientation directed normal to the direction of flow. The answer, as stated earlier in this report, is that the horizontal orientation directed normal to the flow was reasoned to be

TABLE 2

SUSPENDED SAMPLING DATA AND NOZZLE ORIENTATION
FOR A NOZZLE ELEVATION OF 0.15 FOOT ABOVE BOTTOM IN
A WATER DEPTH OF 1.0 FOOT

Sampling Time (sec.)	Sampling Volume (Milliters)	Wave Height (inches)	Weight of Dry Sample (grams)	Concentration By Weight Parts/1000
(a) Nozzle Oriented Normal to Direction Of Flow (Figure 10-a)				
118	7500	9.00	2.34	0.31
121	7400	9.00	1.86	0.25
120	7450	9.00	4.34	0.58
125	7400	9.00	6.88	0.93
136	7450	9.00	2.92	0.39
153	7500	9.00	4.87	0.65
145	7500	9.25	8.91	1.19
140	7450	9.50	4.98	0.67
(b) Nozzle Oriented Normal to Wave Crest And Pointing Seaward (Figure 10-b)				
120	7500	9.25	6.43	0.86
122	7475	9.50	6.13	0.82
122	7400	9.50	7.62	1.03
130	7500	9.50	6.08	0.81
125	7450	9.50	6.45	0.87
145	7400	9.50	7.35	0.99
(c) Nozzle Oriented Normal to Wave Crest And Pointing Shoreward (Figure 10-c)				
132	7450	9.75	8.45	1.13
170	7500	9.75	7.42	0.99
129	7450	10.00	4.39	0.59
179	7450	10.00	4.70	0.63
150	7500	9.75	5.23	0.70
147	7500	9.75	4.48	0.60

the only one which rendered the nozzle intake completely independent of the reversing flow. This statement seems sound, and were the sediment concentration pattern reasonably uniform the orientation directed normal to flow would probably reflect consistent results. However, the restrictions of a non-uniform concentration pattern along the normal to direction of wave travel indicates the compromise orientation along the direction of wave travel may reflect more accurate sampling results.

Sediment Concentration and Temperature Change.. Comparison of the curves of sediment concentration versus depth (Figure 11 a and b) for the water temperatures of 69°F and 47°F respectively, indicated in general, a greater concentration over the range of sampling depths for the lower temperature. This may result from the higher viscosity accompanying the lower temperature. It would seem that the two curves should diverge toward the water surface; that is the colder water or higher viscosity would indicate a relatively greater amount of suspended sediment for those samples nearest the water surface. The higher sediment concentration of the colder water can be partially attributed to a probable lower settling velocity of sediment particles. The lower settling velocity acts to maintain a greater amount of material in suspension. Consequently, for the same wave conditions, there results a greater quantity of suspended load available for transport by littoral forces.

These temperature indications are of a preliminary nature only, very little data having been obtained. However, a similar temperature effect has been previously observed with river flow(3), unidirectional laboratory flume flow(4), and in a laboratory turbulence tank(5). A more intensive study of this indication is currently underway.

Errors of Measurement

In general the errors introduced through physical measurements are not believed serious. The physical measurements refer primarily to those of height of nozzle above bottom. As an example, take the values of concentrations for the mean range of sampling elevations. The maximum probable error for a nozzle elevation displacement of 0.10 foot was 2%, whereas the percent change in concentration for this 0.1 foot elevation displacement averaged from 200 to 500 percent.

CONCLUSIONS

The laboratory tests indicate that a vacuum pump sampler can be adapted to the study of suspended sediment concentrations. The average sediment concentration for a given elevation above the bottom as measured by the sampling equipment and procedure developed appears to be related to the distance between intake nozzle and sand ripple crests.

The moderate success in using the equipment and procedure as developed to obtain the graph (Figure 12) of horizontal distance from crest to trough of a sand ripple versus concentration was in itself a

measure of the reliability of the method. This was the actual reason, aside from its basic research value, for selecting this particular area for measurement and study. The general trend of the curves (Figure 12) could obviously be predicted for uniform sand ripples. This however, made it nonetheless gratifying to have the data from the actual samples collected confirm the prediction that had been made. There was not sufficient data for conclusive proof but the test results indicated considerably higher concentration values in the vicinity of sand ripple crests with lower values nearer the trough. The variability of concentration was more sharply defined at sampling elevations below 0.75D (where D was the water depth) and where most of the material in suspension was concentrated. The water depths used were 1 to 1.2 feet.

The maximum sediment concentration found to exist above the sand ripple crest was greater than the minimum concentration above the sand ripple trough by a factor of four. This was for a constant sampling elevation of 0.1 foot above the bottom (Figure 12). This ratio of maximum to minimum concentration value decreases sharply for higher sampling elevations above the bottom. At elevations of 0.4 to 0.5 foot above the bottom in a water depth of 1.0 foot there was a very minor difference between the concentration values above sand ripple crest and trough.

The principal conclusion we can draw from the explanation is that the relative position of sampling nozzle and local bottom irregularities can greatly influence the sampling results and should be taken into account in the collection of suspended sediment data.

Most of the samples were collected in water at about 20° centigrade, however, in order to investigate experimentally the effect of temperature on suspended sediment, some twenty samples were collected at temperatures of 8° to 9° centigrade (46° to 48°F). Correlation of the suspended sediment data relative to temperature change indicated a general increase in sediment concentration for those samples collected at the lower temperature (Figure 11).

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BEACH EROSION BOARD TECHNICAL REPORT NO. 4

"SHORE PROTECTION PLANNING AND DESIGN"

* * * * * *

Since publication of this report was announced in the last issue of the BULLETIN (Vol. 9, No. 2), 1331 copies have been sold, of the 2,000 copies made available for sale.

The remaining 669 copies are available for purchase from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at $2.25 per copy.

PROGRESS REPORTS ON RESEARCH SPONSORED BY THE BEACH EROSION BOARD

Summaries of progress made during the past year on the several research contracts in force between universities or other institutions and the Beach Erosion Board, together with brief statements as to the status of some research projects being prosecuted in the laboratory of the Beach Erosion Board, are presented below. These summaries supplement and continue those contained in prior issues of the Bulletin.

I. University of California (Scripps Institute of Oceanography) Contract DA-49-005-eng-3. Principal investigator - D. L. Inman

The measurement of sand-level changes by underwater observation of reference rods imbedded in the sand bottom were continued. The total range of sand level at the 18-foot depth station probably exceeded 2 feet, though the reference rods were lost after changes in excess of 0.6 foot were measured. Maximum changes of 0.29, 0.16, and 0.15 foot were measured at stations where the depth was 30, 52, and 70 feet respectively. Significant monthly and seasonal changes occurred. Accuracy of the measurements was stated to be on the order of ±0.05 foot as compared to $\pm\frac{1}{2}$ foot obtained by sonic sounding over the same area. A report by Inman and Rusnak, "Changes in Sand Level On The Beach and Shelf at Ja Jolla" has been issued as Technical Memorandum No. 82 of the Board.

A final report on the orbital velocity measurements, by Inman and Nasu, was prepared as Technical Memorandum No. 79 of the Board. Orbital velocities just seaward of the breaker zone were measured simultaneously with wave pressure records. The observed maximum horizontal velocities were compared with those predicted from solitary wave theory. The agreement was better for longer period waves, but is generally favorable when the ratio of wave height to water depth is greater than about 0.4.

Additional data were gathered describing the physical aspects of ripples generated by shallow water waves. The data indicate that the wave length of the ripples is proportional to the horizontal diameter of the orbit of a water particle near the bottom, when the orbit diameter is less than a limiting value determined by the size of the sand. A report is presently in preparation.

This contract was terminated in March 1956.

II. University of California, Contract DA-49-005-eng-8. Principal Investigator, P. D. Trask

Seasonal sampling of the Pt. Reyes beach has continued, the beach now having been occupied 12 times since June 1953. The sampling has

shown a large areal and seasonal variation in sand and beach parameters. Three preliminary reports have been prepared, one by Trask and Johnson (a summary of the sample data) having been published as Technical Memorandum No. 65 of the Board, one by Chien being abstracted in this issue of the Bulletin, and one by Trask, Johnson, and Scott "Cut and Fill on Pt. Reyes Beach" being given only limited distribution as University of California, Institute Engineering Research, Series 14, Issue 19.

III. Massachusetts Institute of Technology, Contract DA-49-055-eng-16. Principal Investigator, A. T. Ippen

A report, by Ippen and Eagleson, summarizing the work through about June 1955 was issued as Technical Memorandum 63 of the Board. The data indicated a null point to exist separating zones of net onshore and offshore particle motion. Net onshore particle velocities were related to the indicated mass transport velocity. A theoretical analysis was presented yielding a general expression for the net particle velocities in terms of wave and particle parameters, and the boundary layer thickness.

Additional data has now been gathered, particularly on incipient motion, for both smooth and (sand) rough bottoms and for a horizontal as well as sloping bottom. The data obtained seem to indicate particle velocities in excess of the bottom mass transport velocities indicated by Stokes. Instrumentation for measurement of instantaneous boundary layer velocity distributions under waves has been initiated. Rows of tiny hydrogen bubbles are to be released by pulsing a wire probe; synchronized photographs will be taken of the bubbles to indicate speed.

IV. University of California, Contract DA-49-055-eng-17. Principal Investigator, H. A. Einstein

The velocity distribution and turbulence pattern in the turbulent boundary layer near the oscillatory bottom at Reynolds Numbers higher than critical are being investigated - primarily to try to obtain a rational method for predicting bed motion in analogy to the theory for stream traction in unidirectional flow. The flow pattern has been obtained in a general way by introducing dye at various points, and by introducing solid particles having equal density with the water. Difficulties in obtaining exact positions (as two simultaneous views are required) restrict usage of the method however. A recording pitot-tube is now being tried in an attempt to measure more exactly the local velocity in the boundary layer.

V. University of California, Contract DA-49-055-eng-31. Principal Investigator, J. W. Johnson and O. Sibul.

Experimental work on wave run-up and overtopping for levee slopes of 1 on 3 and 1 on 6 was carried out in a wind-wave flume to study the effect of actually generating the waves by wind action rather than

mechanically. Reports by Sibul were issued as Technical Memoranda Nos. 67 and 80 of the Board, and indicate that wind action serves to increase both run-up and overtopping.

VI. University of California. Contract DA-49-055-eng-44. Principal Investigator, J. W. Johnson and R. L. Wiegel

Refracted wave characteristics were measured in the model tank for a series of wave, slope, and depth conditions. Measurements show that wave trains under certain conditions split up into multiple trains, and these conditions are being isolated. Data also shows that in general Snell's law applies, although for certain conditions the waves considerably over-refract. Work in the ripple tank was summarized in a paper by Ralls, "Ripple Tank Study of Wave Refraction" published by the American Society of Civil Engineers.

VII. Agricultural and Mechanical College of Texas. Contract DA-49-055-eng-45. Principal Investigator, C. L. Bretschneider, succeeded by B. W. Wilson

Wave hindcasts for five locations on the Gulf of Mexico coast of the United States were completed for deep water and for four shallow water depths (12, 24, 48, 96 feet). These will appear as Board Technical Memoranda in the near future. Work is continuing on detailed hurricane wave forecasts for 11 hurricanes in the Gulf.

VIII. University of Florida, Contract DA-49-055-eng-55. Principal Investigator, P. Bruun.

A new contract initiated to organize and, where possible, analyse existing data on tidal entrances, principally that data previously collected by the various Corps of Engineer offices. The available data has been collected and sent to Florida for analysis. Considerable progress has been made on the analysis.

IX. New York University, Contract DA-48-055-eng-56. Principal Investigator, S. S. L. Chang

A new contract to provide consultation advice on the use and modification of the wave spectrum analyzer. The analyzer is now being modified to period analysis of laboratory (very short period) waves as well as prototype ocean waves.

X. Agricultural and Mechanical College of Texas, Contract DA-49-055-Civ - eng-56-4. Principal Investigator, R. O. Reid

A new contract to provide analysis of the hurricane surge and associated waves and wind set-up for design of hurricane protection structures in the New England area, primarily Narragansett Bay, New Bedford, and Stamford. A method for predicting hurricane surge was

derived (summarized in a report by Reid published as Technical Memorandum No. 83 of the Board) and elevations were predicted for various points in Narragansett Bay for several hypothetical hurricanes, mainly a transposed Hatteras 1944 hurricane. The method has been calibrated with observations of the 1938 hurricane and with the Narragansett Bay model at the (Vicksburg, Mississippi) Waterways Experiment Station.

XI. Dr. W. C. Krumbein (Consultant)

Statistical study is being made to determine the best sampling procedure to use in determining the characteristics of the littoral material.

Statistical study is also being made to determine methods of establishing the design size characteristics of fill material for artificial fill projects.

XII. Waterways Experiment Station, Vicksburg, Mississippi

(a) Wave Run-up and Overtopping. Principal investigator, R. Y. Hudson.

Testing was completed in the riprap-faced seawall with a seaside slope of 1 on $1\frac{1}{2}$. A report by Saville summarizing the data taken over the past three years was issued as Technical Memorandum No. 64 of the Board.

(b) Study of Effect of Inlets on Adjacent Beaches. Principal investigator, H. B. Simmons

The inlet condition resulting from 300 tidal cycles with the shallow lagoon was tested with a storm tide superimposed on the regular tide. Tidal elevations were raised gradually over seven tidal cycles to a maximum and then reduced to normal over the succeeding three tidal cycles. A break-through of the migrating (above water) bar occurred with the storm tide and, once a channel was fairly well defined, it migrated downbeach.

(c) Study of Durability of Concrete in Shore Structures. Principal investigator, B. Mather and T. B. Kennedy

A study is being made summarizing the factors affecting the durability of concrete in shore structures, the results of tests and investigations of durability of concrete specimens in coastal exposures, and selected service records of specific prototype concrete structures.

A report is in preparation.

XIII. Beach Erosion Board

The Research program of the Board, of which the above summarized contract work forms a part, is carried on for the most part in the Board's

own facilities. Many of these projects have been described in previous numbers of the Bulletin, and progress on the major projects during the last year is summarized below.

(a) Measurement of Suspended Material in Laboratory Wave Tanks

Development of a laboratory sampler and sampling techniques is discussed in a report by Fairchild in this Bulletin. For samples taken relatively close to the sand bottom, the exact positioning of the sampler intake relative to the ripple crest or trough appear to be quite important. Additional testing on the effect of water temperature is underway, and indicates that considerably greater amount of material are placed in suspension in colder water. One test on resultant equilibrium profile under two different temperature conditions indicates that the same eventual profile results, but adjustment occurs faster with the colder water.

(b) Laboratory Study of Effects of Groin Field on Littoral Drift Passing Field

Certain preliminary tests were run in the Shore Processes Test Basin to determine probable range of sand transport rates. A new and more precise sand trapping and measuring system is being installed, with an eductor system to transfer the trapped material back to the upbeach end of the test area.

(c) Equilibrium Profile Tests

Temperature effects are discussed under (a) above. Two tests have been run in the prototype tank involving waves of 2.5 and 4 feet in height with 1 to 10 and 1 to 15 scale models run in small flumes utilizing the same sand and a 1 on 15 initial slope. The tests indicate that the sometimes used "critical" value of wave steepness of 0.02 to 0.025 found in some small model tanks as the dividing point between beach erosion and beach accretion for comparable sands possibly does not apply for larger waves; that with large waves, waves having steepness considerably less than 0.02 may erode rather than accrete the beach. This possibility has been previously indicated by Rector reporting (Board T.M. #41) on small scale tests where it was shown that the "critical" steepness possibly also depended on a sand-wave size parameter (as grain diameter - wave height ratio).

(d) Forces on Piling

A 12-inch pile with a 3-foot sensitive (instrumented) section has been mounted in the prototype tank, and force measurements are being taken for depth and wave conditions -- including breakers.

(e) Wave Forecasting

Methods of forecasting for shallow water generation (as the continental shelf of the Gulf of Mexico) have been developed to include the effect of friction, refraction and shoaling, and a report will be issued in the next year. An analysis and comparison of the Bretschneider revised Sverdrup-Münk method with the spectrum method developed by Neumann and Pierson is being made.

(f) Maximum Water Stage Level Recorder

A maximum water stage level recorder similar to that used by the Geological Survey for determining maximum river flood stages was tested to determine the effect of superficial wave action in displacing the reading. A modification to the gage, essentially choking down the inlet area to the gage, resulted in excess rises inside the gage (errors) being less than one tenth of the wave height, never amounting to greater than 0.3 foot. Such a gage could possibly be used economically around estuaries to obtain maximum hurricane tide levels.

(g) Wave Run-up

Testing of eight smooth slopes ranging in steepness from 1 on 30 to a vertical wall has been completed and a report by Saville "Wave Run-up on Structures" published by the American Society of Civil Engineers. Most of these slopes have been tested for the effect of roughness on wave run-up by roughening the slopes with five sand sizes ranging from 0.2 mm. to 6.50 mm. in median diameter. About half of the slopes have been tested for the effect of permeability on wave run-up for the same sands used in the roughness test. The data show that slope roughness and permeability reduce wave run-up considerably below that observed for smooth slopes.

(h) Study of Sand By-Passing Operation at Port Hueneme

As a continuation of the observational program, a survey of the area was made in June 1956. The data taken before, during, and after the Port Hueneme dredging project to June 1955 has been analysed and a report has been prepared. The report indicates that if sand movement continues at the rates indicated, the dredged area will be filled by June 1957 and the erosion downbeach will again reach the Pt. Mugu area by June 1959. Serious erosion is already occurring immediately southeast of the east jetty.

(i) Recorded Wave Characteristics

Routine compilations were made of recorded wave characteristics at: Cape Henry, Virginia; Huntington Beach, California; Daytona Beach, Florida; Clearwater Beach, Florida; Palm Beach, Florida; Evanston, Illinois; Bay Marchand, Louisiana.

(j) Sand Movement Study at Moriches Inlet, Long Island, New York

A study of sand movement through an inlet and along the adjacent shore based on previously collected field data is underway.

(k) Reexamination of Artificial Fill Projects

Continuing program of reexamination of artificially nourished beaches to determine the effectiveness of the fill material within the beach zones, and to better establish the factors upon which to base the design characteristics of fill material. Study of fill at Ocean City, New Jersey, was reported on by Watts in T. M. #77. Reexamination is now underway of projects at Virginia Beach, Virginia and Plum Island, Massachusetts.

(l) Technical Report No. 4, "Shore Protection Planning and Design"

Continuing study to improve and supplement present chapters of this publication. A report on "Factors Affecting the Economic Life of Timber in Coastal Structures" by Jachowski was published as T. M. #66. General corrections and addenda to T.R. No. 4 have been completed and will be disseminated when printed.

(m) Regional Studies

Data on the geomorphology and characteristics of littoral materials were compiled for the Delaware coast. Similar material is being gathered for the south shore of Long Island.

XIV. <u>New Equipment</u>

It may also be of interest to note that the prototype wave tank was completed and put into operation in the Spring. The tank is of reinforced concrete, and is 635 feet long, 20 feet deep and 15 feet wide. The wave-generating mechanism provided for the wave tank consists of a vertical bulkhead 15 feet wide and 23 feet high, connected to a carriage. The carriage moves back and forth on rails mounted on each wall of the tank. Top rails are also required to prevent lifting of the carriage from the rails during operation. The back and forth motion is transmitted to the bulkhead and carriage by two arms, 43 feet-9 inches in length, connected to two driving discs. Each disc is 19 feet in diameter and weighs 14 tons. The discs are driven through a train of gears by a 510-horsepower, 2,300-volt synchronous motor. The motor has a constant speed of 1,200 revolutions per minute. Four interchangeable pairs of gears are provided to permit variations in the speed of the discs. The alternate gearing will allow wave periods of approximately 5.60, 7.87, 11.32, and 16.01 seconds. The lengths of stroke of the bulkhead movement can be varied from 2 feet to 17 feet-6 inches by changing the eccentric setting of the connecting arms on the driving discs. The stroke setting may be varied in 3-inch increments through the range from 2 to 8 feet and in 6-inch increments from 8 feet to 17 feet-6 inches.

When the connecting arm eccentric setting on the disc is changed, the disc is rebalanced with counterweights. The counterweights required for balancing range from 378 pounds for each disc for the shortest (2-foot) stroke to 3,820 pounds for each disc for the longest (17-foot-6 inch) stroke.

Normally a depth of 15 feet of water will be maintained in the tank. Desired wave heights are obtained by controlling the combination of stroke and period of the bulkhead movement, but the height may also be controlled by increasing or decreasing the depth of water in the tank. Wave heights up to 6 feet in the deep portion of the tank can be generated, with breaking heights exceeding (for some conditions) 8 feet. The tank is presently being used for studies on forces on piling, and equilibrium beach profiles.

BEACH EROSION STUDIES

Beach erosion control studies of specific localities are usually made by the Corps of Engineers in cooperation with appropriate agencies of the various States by authority of Section 2 of the River and Harbor Act approved 3 July 1930. By executive ruling the costs of these studies are divided equally between the United States and the cooperating agencies. Information concerning the initiation of a cooperative study may be obtained from any District or Division Engineer of the Corps of Engineers. After a report on a cooperative study has been transmitted to Congress, a summary thereof is included in the next issue of this Bulletin. Summaries of reports transmitted to Congress since the last issue of the Bulletin and lists of completed and authorized cooperative studies follow.

SUMMARIES OF REPORTS TRANSMITTED TO CONGRESS

GRAND ISLE, LOUISIANA

Grand Isle, located on the Gulf Coast of Louisiana, has a maximum width of about 3/4 mile and length of about 7.5 miles extending in a general northeast to southwest direction between Barataria and Caminada Passes. The island is low with an elevation of 3 to 5 feet above mean low water in the central section and a maximum height of 6 feet at dunes along the shore near the western end. The island is located in Jefferson Parish about 60 miles south of New Orleans and 50 miles northwest of Southwest Pass of the Mississippi River. Grand Isle is used as a summer resort, also for seafood and oil production activities. Its permanent population is 1,190, but it is estimated that more than 3,000 people visit the beach during summer week ends. Although not highly developed at present, it is increasing in popularity for recreational use. The entire shore is privately owned.

Tides in the area are semi-diurnal, the mean range being 1.1 feet and the spring range 1.7 feet. In 1947 the tide reached a height of 3.4 feet above high water. The Gulf shore of the island is exposed to waves from the southeast, south and southwest. As Southwest Pass limits the fetch to the east and southeast, waves approaching the shore are more likely to have a component tending to cause eastward littoral drift. Predominant drift in that direction is indicated by accretion west of existing groins and erosion of the shore east thereof.

The Gulf shore of Grand Isle has had a history of intermittent erosion and accretion. In earlier years there was erosion on the western half and accretion on the eastern half of the island. In recent years accretion has occurred at the western end and erosion on the central and eastern sections. In 1951-52 the State built four groins near the western end and ten near the middle of the island for protection of the highway. These structures have caused accretion at the west end

of each group, but have aggravated erosion east of each group. Local interests desire a plan for protection and stabilization of the shore.

The Division and District Engineers and the Beach Erosion Board concluded that the most suitable minimum plan of protection comprised placement of approximately 1,000,000 cubic yards of suitable material in two stockpiles to fill both groups of existing groins and adjacent eroding areas east thereof, and construction of a single jetty at the eastern end of the island near Barataria Pass. The maintenance program for providing continued stability to the shore consisted of periodic subsequent replenishment of these stockpiles or placement of equivalent fill at locations as required by shifting of the critical areas of erosion, the estimated average annual requirement being 100,000 cubic yards. They further concluded that the shore being privately owned, the project is ineligible for Federal aid under existing policy and recommended that no project be adopted by the United States for stabilization and protection of the Gulf shore of Grand Isle at this time. They further recommended that protective measures that may be undertaken by local interests, based on their determination of economic justification, be accomplished in accordance with plans and methods proposed in the report.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

FAIR HAVEN BEACH STATE PARK, NEW YORK

Fair Haven Beach State Park comprises about 1½ miles of shore on the southern shore near the east end of Lake Ontario. It is located in the Town of Sterling, in Cayuga County, about 15 miles southwest of Oswego Harbor. It is conveniently located with respect to the population centers of Oswego, Syracuse, Auburn, and Geneva, which have a combined population of about 300,000. The park has been developed with cottages, camping facilities, bathing beach, picnicking and parking areas. Fair Haven Beach State Park lies immediately east of the jettied entrance at Little Sodus Bay. The jetties and a breakwater connecting the east jetty to the park shore were built by the United States under a navigation project. The shore of the park comprises bluffs of glacial till alternating with barrier beaches across the drainage courses. The westerly beach is a barrier bar fronting Sterling Pond. The outlet to the pond has been protected by the State by low training walls.

Lake Ontario is about 190 miles long and 50 miles wide. The mean lake level for the months of March to December is about 2 feet above the established low water datum. The highest lake stage and the highest monthly mean recorded at Oswego, New York, are respectively about 6.2 and 5.3 feet above low water datum. Storms cause changes in lake levels as winds move the water toward the ends of the lake. The design lake stage is 6 feet above low water datum. Of winds which generate waves affecting the area, those from the west have the greatest

fetch, about 150 miles. During severe storms with a frequency of about once a year, waves may range up to 11 feet in height in deep water, but ordinarily waves of this height would break before reaching shore structures. Storms waves which cause the greatest movement of beach material are those from the west. Although the predominant direction of littoral drift is eastward, no material reaches the shore past the jettied entrance to Little Sodus Bay. Sterling Creek flows through swamps and furnishes no material to the shore.

The Division and District Engineers and the Beach Erosion Board concluded that the most suitable plan for the protection of Fair Haven Beach State Park consists of:

a. For the westerly beach, modification of the Sterling Creek outlet and the existing concrete groin, placement of approximately 71,000 cubic yards of sand fill, and construction of a new groin at the westerly limit of the beach;

b. For the bluff area, stone revetment of approximately 2,700 feet of shore; and

c. For the easterly beach, construction of one groin and placement of approximately 27,500 cubic yards of sand fill.

They found that protection and improvement of the westerly beach at Fair Haven Beach State Park were justified by evaluated benefits, and recommended, subject to certain conditions, adoption of a project by the United States authorizing Federal participation in an amount equal to one-third of the first cost of the plan for the protection and improvement of the westerly beach area. They further recommended that the State consider adoption of the plans for the bluff and easterly beach areas, but recommended no Federal project for those areas.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

HAMLIN BEACH STATE PARK, NEW YORK

Hamlin Beach State Park comprises about $2\frac{1}{4}$ miles of shore in the central part of the southern shore of Lake Ontario. It is located in the town of Hamlin in Monroe, County, about 20 miles west of Rochester. It is thus conveniently located with respect to that population center, with a population of 332,000. The combined population of tributary counties is about 750,000, exclusive of the city of Buffalo with a population of 580,000. Although Buffalo is somewhat beyond a 50-mile radius, it contributes a considerable number of visitors annually to Hamlin Beach. The park has been developed with bathing beach, picnicking and parking areas. It is remote from potential sources of pollution likely to endanger the health of bathers. The western 0.3 mile of the shore of the park consists of bluffs of fine sand and clay gradually decreasing in height from a maximum of about 30 feet. The remainder of the shore is relatively flat with elevations of 7 to 15 feet above the lake. It consists of low bluffs alternating with barrier beaches at the drainage courses. Yanty Creek, the largest creek, flows through

swamps and furnishes no material to the shore. It has been proposed as a site for a small-boat harbor.

Lake Ontario is about 190 miles long and 50 miles wide. The mean lake level for the months of March to December is about 2 feet above the established low water datum. The highest lake stage and the highest monthly mean recorded at Oswego, New York, are respectively about 6.2 and 5.3 feet above low water datum. Of winds which generate waves affecting the area, those from the west and east have the greatest fetch, about 90 miles. However, the wind movement and duration from the west and northwest considerably exceed those from the east and northeast, consequently, waves which cause the greatest movement of beach material are those from the west and northwest. During severe storms, which occur with a frequency of about once a year, waves may range up to 12 feet in height in deep water, but ordinarily waves of this height would break before reaching shore structures.

The Division and District Engineers and the Beach Erosion Board concluded that the most suitable plans to provide the beach areas and the protection for remaining areas of Hamlin Beach State Park consist of:

a. For the westerly beach area, construction of four new groins; modification of three existing groins, grading the bluffs, and placement of approximately 217,000 cubic yards of sand fill;

b. For the central beach area, construction of three new groins modification of one existing groin, placement of approximately 123,000 cubic yards of sand fill, and construction of approximately 500 feet of stone and concrete revetment;

c. For the easterly beach area, construction of three new groins, placement of approximately 51,000 cubic yards of sand fill, and construction of approximately 2,600 feet of stone revetment.

They found that the restoration and protection of the westerly beach at Hamlin Beach State Park were warranted by evaluated benefits, and that Federal participation in the first cost thereof is justified. They recommended adoption of a project by the United States authorizing Federal participation, subject to certain conditions, in an amount equal to one-third of the first cost of the plan for the restoration and protection of the westerly beach area. They further recommended that the State of New York consider adopting the foregoing plans for the central and east beach areas and that no Federal project be adopted at this time for those areas, but that further consideration be given to the advisability of Federal participation in such a project if requested by the cooperating agency at such time as plans for park development are further advanced and the benefits are determinable.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

The city of Kenosha is located in Kenosha County, about 30 miles south of Milwaukee, Wisconsin, and 50 miles north of Chicago Illinois. It has a shore frontage on Lake Michigan of about 4½ miles, of which about 75 percent comprises parks owned by the city. The remainder of the above frontage is used for industrial and residential purposes. The city and county have populations of 54,400 and 75,200 respectively. North of the harbor entrance, the shore is backed by low bluffs with a maximum height of about 25 feet. Beaches range from 0 to 180 feet in width. A groin system has helped accumulate a beach at one park and another beach has accumulated at the north jetty of the harbor. The harbor works consist of parallel jetties and a detached breakwater to protect the entrance. South of the harbor the shore has partly been formed by breakwaters and fills. The remainder has been protected by walls, bulkheads and revetments.

Lake Michigan is over 300 miles long and about 80 miles wide. The mean lake level for the period 1900 to 1953 was 1.4 feet above the established low water datum. The highest monthly mean recorded was 3.8 feet above mean lake level and 5.2 feet above low water datum. Short period fluctuations up to about 2 feet are caused by winds and differences in barometric pressure. The design lake stage is 5 feet above low water datum. Of winds which generate waves affecting the area, those from the north and northeast have the greatest fetch, about 260 miles. During severe storms with a frequency of about once a year, waves may range up to 13 feet in height in deep water, but ordinarily waves of this height would break before reaching shore structures. Storm waves which cause the greatest movement of beach material are those from the northeast. The predominant direction of littoral drift is southward, but practically no material moves along the shore south of the jettied entrance to Kenosha Harbor. The small streams entering the lake in Kenosha County furnish little material to the shore.

The Division and District Engineers and the Beach Erosion Board concluded that the most suitable plans for preventing erosion and stabilizing the shores of Kenosha consist of the following:

Alford Park	- Maintenance of existing groin system.
Pennoyer Park	- Stone revetment for 650-foot reach of shore.
Lake View Park	- Reconstruct 1,620 feet of old breakwater.
Simmons Island Park	- Reconstruct and extend southerly the rubble mound to protect 425 feet of shore. An alternative plan of steel sheet pile wall may be used for the southerly 125 feet of this reach.

Simmons Manufacturing Company and Lake Front Park	- Reconstruct 2,570 feet of breakwater.
Eichelman Park	- Raise 250 feet of low concrete seawall and riprap entire length of 748 feet of seawall. Reconstruct 670 feet of offshore breakwater.
Eichelman Park to 75th Street	- Maintenance and reconstruction of existing stone revetment.
Southport Park	- Stone revetment for 2,090 feet of shore. An alternative plan provides for reconstruction of groin system and protective beach of artificial fill for northerly 1,700 feet of problem area and revetment of stone for southerly 390 feet.

They found evaluated benefits insufficient to justify Federal participation in the costs of additional protection for the shores of Kenosha, and recommended that no projects be adopted by the United States at this time for protection of shores of Lake Michigan within the city of Kenosha. They further recommended that protective measures which may be undertaken by local interests, in accordance with their own determination of economic justification, be accomplished in accordance with plans proposed in this report.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

MANITOWOC COUNTY FROM TWO RIVERS TO MANITOWOC, WISCONSIN

The study area comprises a $9\frac{1}{2}$-mile section of the shore of Manitowoc County from the north limit of Two Rivers to the south limit of Manitowoc. It is located about 70 miles north of Milwaukee, Wisconsin. About 77 percent of the shore frontage of the study area is publicly owned. The remainder of the above frontage is used for industrial and residential purposes. The county has a population of about 67,000. The exposed shore frontage of the study area is divided into three sections by the Federally improved harbors of Two Rivers and Manitowoc. North of Two Rivers Harbor the shore is backed by low banks averaging about 6 feet in height above mean lake level. Beaches are 50 to 100 feet in width at mean lake level. The entrance to Two Rivers Harbor is protected by parallel jetties (locally known as piers) through which East and West Twin Rivers discharge. Between the harbors the shore consists of low bluffs ranging in height to a maximum of about 25 feet above mean lake level. Beaches are generally narrow at mean lake stage. The outer basin of Manitowoc Harbor at the mouth of Manitowoc River is formed by two shore-connected converging breakwaters. South of the harbor the elevations of the shore bluffs range from about 5 to 50 feet above mean lake level.

Beach widths in this section range up to a maximum of about 75 feet at mean lake stage.

Lake Michigan is over 300 miles long and about 80 miles wide. The mean lake level for the 94-year period of record is 2.1 feet above the established low water datum. The highest monthly mean recorded was 3.1 feet above mean lake level and 5.2 feet above low water datum. Short period fluctuations up to about 1.5 feet are caused by winds and differences in barometric pressures. The design lake stage is 5 feet above low water datum. Of winds which generate waves affecting the area, those from the north-northeast have the greatest fetch, about 175 miles, but those from the south-southeast have a fetch of about 170 miles. During severe storms with a frequency of about once a year, waves may range up to 11 feet in height in deep water, but ordinarily waves of this height break before reaching shore structures. Waves from both the northeast and southeast quadrants cause movement of beach material, but there appears to be little net transport in either direction between Two Rivers and Manitowoc Harbors. North of Two Rivers Harbor the predominant direction of littoral transport is southward. Immediately south of Manitowoc Harbor the direction of littoral movement alternates, but there appears to be a slight northward predominance of littoral transport. The small streams entering the lake in Manitowoc County furnish little material to the shore.

The Division and District Engineers and the Beach Erosion Board concluded that the most suitable plans of preventing erosion and stabilizing the shores of the study area consist of stone revetments or rubble mounds and that beach fill plans are practicable south of Manitowoc Harbor but economically unjustified. They found that protection of 9,550 feet of shore along State Highway 42 and Cleveland Avenue between Two Rivers and Waldo Avenue in Manitowoc by stone revetment at the toe of the bluff was warranted by evaluated benefits. They therefore recommended adoption of a Federal project authorizing Federal participation therein to the extent of one-third of the first cost, subject to certain conditions. They further found that protection of the remaining frontage of the study area was not justified by evaluated benefits, but recommended that protective measures which may be undertaken by local interests for other sections of shore, based on their own determination of economic justification, be accomplished in accordance with plans proposed in this report.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

The study area comprises the Atlantic Ocean shore line of New Jersey from the north end of Sandy Hook to Barnegat Inlet, a length of about 51 miles. This stretch of shore lies in Monmouth and Ocean Counties. The principal communities along this shore are Sea Bright, Monmouth Beach, Long Branch, Asbury Park, Point Pleasant Beach, Seaside Heights, and Seaside Park. The permanent population of the municipalities within the study area is 90,000. The estimated peak summer population is 600,000. The tributary area from which visitors are principally drawn comprises northern New Jersey and portions of southern New York with a total population of about 6,000,000. However, visitors come from all States in the Union. The Sandy Hook section of the study area, 6 miles in length, is owned by the United States and is occupied by the Fort Hancock Reservation. Of the remaining 45 miles of ocean frontage, about 55 percent is publicly owned and 45 percent privately owned. The southern portion extending about 10 miles north from Barnegat Inlet is known as Island Beach. It has been acquired by the State for park purposes.

On the basis of present need for shore restoration and protection, the shore is divided into five sections as follows:

a. Sandy Hook Section - For purposes of this report the Sandy Hook section comprises only the Government Reservation of Fort Hancock, the northern 6 miles of the sand spit extending 11 miles from the headland at Monmouth Beach. The surface consists principally of low sand dunes. The elevation is about 5 to 10 feet above mean sea level. There is no immediate need for plans of protection of the Government Reservation. If protection becomes necessary, plans therefor may be obtained by the using agency under other available authority.

b. Sea Bright-Ocean Township Section - This section includes Long Branch, the largest city in the study area. It is about 12 miles long comprising 5 miles of sand spit and 7 miles of headland. The bluff adjoining the ocean has an elevation of 10 to 25 feet above mean sea level. The section has had practically a continuous history of erosion. Seawalls, bulkheads and groins have partially stabilized the shore line but the upland is still subject to storm damage. A few beaches exists at the south sides of the larger groins. The section is extensively developed as a summer resort area, but the economy of this section has been adversely affected by lack of protective and recreational beaches. About 23 percent of the shore frontage is publicly owned.

c. Asbury Park-Manasquan Section - This 9-mile section is part of the headland portion of the coast with bluff elevations of 10 to 15 feet above mean sea level. It includes the city of Asbury Park and an intensively developed portion of the study area. As a result of extensive groin construction, protective and recreational beaches have been retained to some extent; however, their widths are generally

insufficient for these purposes. About 96 percent of the frontage is publicly owned. Shark River Inlet is included in this section. Another relatively small inlet, Manasquan Inlet, is the south limit of the section. Both have been improved for navigation, including twin jetties.

d. Point Pleasant Beach to Seaside Park - This section is about 14 miles long, including a ½-mile section in Berkeley Township adjoining Seaside Park to the south. It includes 3 miles of headland south to Bay Head. The remainder is in the form of barrier beach with widths of 500 feet to 1 mile and elevations up to 12 feet above mean sea level, except for dunes along the shore. Although farther from population centers and not as intensively developed as the sections to the north, the section is popular because of its better beaches. In addition to communities mentioned above, the principal summer colonies are Mantoloking and Lavallette, but the remainder is being steadily developed for private recreational use. The shore has been more stable than that to the north, and few protective measures have been undertaken. However, the width of protective beach has recently been depleted at Bay Head and Lavallette, and the State has partially restored the beach at Lavallette by artificial placement of sand since the November 1953 storm. A program is needed to provide for continued stability of the shore in this section. About 25 percent of the frontage is publicly owned.

e. Island Beach - This section, nearly 10 miles in length, is a separate entity with direction of drift opposite to that in the remainder of the study area, and hence affected only to a minor degree by protective measures in the adjoining section. The proposal for early development of the section as a State Park necessitates that the past history of shore line recession be considered in planning such development. Barnegat Inlet, a large inlet at the south limit of the study area, is Federally improved for small boat navigation.

Tides in the area are semi-diurnal, the mean range being 4.5 feet at Sandy Hook and 4 feet at Barnegat Inlet. The spring range is about 5 feet. The maximum tide of record at Sandy Hook, 9.1 feet above mean sea level, occurred during the storm of November 1953. The shore of the study area is exposed to waves from the northeast, east and southeast. The fetch to the east and southeast is unlimited, but Long Island shields the area from waves from the northeast. As a result the predominant direction of littoral drift is northward from a nodal region between Bay Head and Barnegat.

The District Engineer developed a plan for protecting and stabilizing the shores from Sea Bright to Seaside Park. The District Engineer made an economic analysis of the proposed work and found that the benefits from prevention of damages, increased earning power of property, reduction in maintenance costs of existing structures, and recreational benefits from additional beach use would amply justify the work. The Division and

District Engineers and the Beach Erosion Board concluded that the public interest in the improvements was sufficient to warrant Federal assistance to the extent of one-third of the costs applicable to the publicly owned portions of the shore, in accordance with the policy established by Public Law 727, 79th Congress. They recommended adoption of a project by the United States authorizing Federal participation, subject to certain conditions, to the extent of one-third of the first costs of measures for the restoration and protection of the publicly owned portions of the shore from Sea Bright to Seaside Park under plans comprising artificial placement of approximately 13,901,000 cubic yards of sand on the shore to restore a protective beach generally 100 feet wide at elevation 10 feet above mean low water, supplemented by groin construction in the Sea Bright-Ocean Township section. The Beach Erosion Board emphasized that periodic replenishment operations as required are an essential part of the plan for maintaining the protective and recreational beaches, and providing for continued stability of the shore. The Board considered the Sandy Hook section to be an integral part of the shores to the south with respect to littoral processes, even though it had been omitted from the project because its protection did not appear justified on the basis of present use. The Sandy Hook shore has been eroding slowly north of the seawall at the south end of the Government Reservation. That erosion may be expected to continue, and if local interests fail to provide artificial nourishment in conjunction with construction of groins, as recommended on adjoining shores to the south, erosion of Sandy Hook will be accelerated.

The Chief of Engineers concurred generally in the views and recommendations of the Beach Erosion Board.

BRADDOCK BAY STATE PARK, NEW YORK

Braddock Bay State Park includes about one mile of publicly owned shore in the central part of the southern shore of Lake Ontario. It is located in the Town of Greece in Monroe County, about 7 miles west of Rochester. It is thus conveniently located with respect to that center, which has a population of 332,000. The combined population of tributary counties is about 750,000. The State plans eventually to develop the park with bathing beach, picnicking and parking areas and a small-boat harbor. Pollution of the lake waters in the vicinity of Braddock Bay is not considered sufficient to endanger the health of bathers. The lake frontage of the study area is a low barrier beach located between two low headlands about one mile apart. An outlet through the beach was formerly narrower, but much of the barrier is now a submerged bar and the main opening is about 1,600 feet wide. Two creeks having a combined drainage area of about 77 square miles empty through the bay.

Lake Ontario is about 190 miles long and 50 miles wide. The mean lake level for the months of March to December is about 2 feet above the established low water datum (244.0 feet above the mean tide at New York City). The highest lake stage and the highest monthly mean recorded

at Oswego, New York, are respectively about 6.2 and 5.3 feet above low water datum. Storms cause changes in lake levels as winds move the water toward the ends of the lake. The design lake stage is 6 feet above low water datum. Of winds which generate waves affecting the area, those from the northwest and northeast have the greatest fetches, about 55 and 60 miles respectively. However, the wind movement and duration from the west and northwest considerably exceed those from the east and northeast, consequently waves which cause the greatest movement of beach material in the vicinity are those from the west and northwest. During severe storms which occur with a frequency of about once a year, waves in deep water may range up to 12 feet in height with periods of 8 to 9 seconds, but ordinarily waves of this height would break before reaching shore structures.

The Division and District Engineers and the Beach Erosion Board concluded that the most suitable plan of restoration and protection of the shore at Braddock Bay State Park consists of:

a. Sand beach fill: Construction of a beach 4,800 feet long, between Braddock Point and Braddock Heights, with berm 50 feet wide at elevation 254.5 feet, by placement of approximately 440,000 cubic yards of suitable sand;

b. Outlet jetties: Construction of two jetties approximately 650 feet long and 150 feet apart at the west end of the beach; and

c. Alteration of existing stone groin at easterly park limit: Raising of the inner 110 feet of the existing stone groin to elevation 254.5 feet, and raising an additional 120 feet to a uniform slope from that elevation to the top of the existing groin.

As the State's primary interest at the time was in development of general plans for future construction of a beach to protect park improvements which are unlikely to be undertaken in the near future, no economic analysis to determine justification for Federal participation in the project was made by the District Engineer. However, the Division and District Engineers and the Beach Erosion Board all recommended that the State of New York consider adoption of the foregoing plan of protection and improvement of the shore frontage. They further recommended that no project be adopted at this time authorizing Federal participation in the cost of protection and improvement of Braddock Bay State Park.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

PERDIDO PASS (ALABAMA POINT), ALABAMA

The study area comprises the Gulf and Perdido Pass shore lines of Alabama Point, located on the Gulf of Mexico about 40 miles east of the entrance to Mobile Bay. The point is owned by the State. Florida Point,

on the east side of the pass has been a part of Florida, but would be transferred to Alabama under an agreement ratified by both legislatures, subject to approval by Congress. The State of Alabama proposes to acquire about 2 miles of Florida Point for development as a recreational area. The immediate tributary area is sparsely settled, but a population of about 400,000 is located within a 50-miles radius, including the city of Mobile. The Gulf shore is increasing in importance as a recreational area.

The low mainland bluffs are composed of erodible materials which have furnished large quantities of sand to the shore. As a result embayments are separated from the Gulf by extensive barrier beaches. The barrier beach across Perdido Bay is about 8 miles in length broken only by the narrow pass. Ono Island about 5 miles in length is the remains of an earlier barrier lying behind the present beach. Tides in the area are diurnal, the mean diurnal range being 1.3 feet. The principal wave action is from the southeast. As indicated by inlet migration, the predominant littoral drift is westward. There is no evidence of important reversals in direction. The estimated volume of westward drift is 200,000 cubic yards annually. Erosion of Alabama Point is caused by waves entering from the Gulf and strong tidal currents forced against the point by westward elongation of Florida Point.

The District Engineer developed plans for protecting Alabama Point and stabilizing Perdido Pass. The alternative methods are (a) a single jetty extending southwestward from Florida Point, (b) an offshore breakwater off Florida Point, or (c) a steel sheet pile bulkhead around Alabama Point. The Division and District Engineers concluded that the most suitable plan for the intended purpose considering all costs, advantages and disadvantages consists of a single rubble-mound jetty about 3,100 feet long extending southwestward from Florida Point. They concluded further that the plan was not justified by evaluated benefits, but that intangible and secondary benefits not sufficient to warrant Federal participation may be considered by the State in its own evaluation of the plan and in comparison with the alternative of bridging the pass farther inland. Accordingly they recommended that no Federal project for stabilization of the pass in the interest of protecting Alabama Point be authorized at this time.

The Beach Erosion Board concurred in the views and recommendations of the Division and District Engineers but desired to emphasize the fact that stabilization of a migrating inlet is a difficult and costly problem, and that the jetty plan considered most suitable by the District Engineer must include a dredging program to supply material to the downdrift shore and to prevent deterioration of the channel.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

OCEANSIDE, OCEAN BEACH, IMPERIAL BEACH AND CORONADO, SAN DIEGO COUNTY, CALIFORNIA

San Diego County is in Southern California immediately north of the Mexican boundary. Its Pacific Ocean shore line, extending in a general north-south direction, is about 70 miles long. The coastal area consists generally of a series of long narrow beaches backed by steep hills and bluffs, except that at Mission and San Diego Bays it consists of low sandy peninsulas separating the ocean from those bays. The coastal area of the northern half of the county is drained by short steep intermittent streams. In the southern half, the drainage was formerly into Mission and San Diego Bays, except for the Tia Juana River which discharges into the ocean about 1½ miles north of the Mexican border. Since 1951 San Diego River discharges directly into the ocean through a separate flood channel adjacent to the entrance to Mission Bay.

The City of Oceanside with a population of about 18,400 is in the northern part of San Diego County about 18 miles south of the north county line. It has an ocean frontage of about 3 miles, characterized by bluffs 20 to 40 feet high fronted by a narrow beach. The shore frontage requiring protection, about 10,000 feet in length, is owned by the city. The center of Oceanside is about 2 miles south of the entrance to the Federal harbor at Camp Pendleton. That harbor consists of an inner basin about 33 acres in area and 20 feet in depth. The entrance is protected by converging jetties. Due to littoral drift and small tidal prism, the maintenance requirements of the entrance channel have been excessive and the entrance is now practically closed. The dominant littoral drift is from north to south. The problem at Oceanside is one of shore recession over the past 13 years due to impoundment of littoral drift at the harbor jetties which were completed in 1943. The rate of recession is probably decreasing as the limit of impounding capacity of the jetties is approached, but the city frontage is presently without an adequate protective and recreational beach and shore front property is endangered. Erosion is continuing, though at a diminishing rate. Restoration and maintenance of the beach are desired.

Ocean Beach, a part of the City of San Diego, is a residential community with a population of about 34,800. It is located on the ocean about 5 miles west of the city's business center. Most of the community is on high ground with a beach about ½ mile long lying between the Mission Bay-San Diego River jetties on the north and Sunset Cliffs on the south. The shore is owned by the City of San Diego. Since construction of the Mission Bay jetties in 1948 to 1950, Ocean Beach has become a pocket beach between Sunset Cliffs and those jetties. The beach has reoriented, the material moving northward to form a beach at the northern portion of the pocket and across the mouth of San Diego River. The southern portion lacks an adequate protective or recreational beach. Local interests desire restoration and stabilization of this beach.

Under the Federal navigation project for Mission Bay, dredging of the entrance channel and deposition of spoil on Ocean Beach at no cost to any project for shore protection has recently been completed. In order to obtain the maximum benefits from this beach fill, it was essential that the groin considered in this report be built prior to northward shifting of the fill by summer waves. Accordingly local interests proceeded with construction of the groin at Cape May Avenue and the shore protection project for Ocean Beach, as recommended by the reporting officers, is now substantially complete.

Imperial Beach is a small residential community with a population of 285. It lies on the ocean shore about 3.5 miles north of the Mexican border. The improved frontage where protection is required has a narrow sandy recreational beach about 6,300 feet long. The southerly section, 5,700 feet long, is owned by San Diego County and the northerly section, 600 feet long, is Federally owned and occupied by the U. S. Naval Radio Station. The ocean waters in the Imperial Beach area are contaminated by sewage discharged by the international sewer 0.7 mile north of the international boundary. Efforts are being made to correct this condition. At Imperial Beach the problem is one of gradual erosion which has reduced the beach width. The erosion has been caused by inadequate natural supply of beach material. Control structures which have reduced flood flows in the lower Tia Juana River, coupled with lack of floods since 1928,have greatly reduced the volume of detritus brought to the shore. The lack of protective beach has exposed upland public and private property to damage. Restoration of the former width of protective and recreational beach and prevention of further erosion are desired.

Coronado is a residential city on the peninsula separating San Diego Bay from the ocean. The present problem area is at Bay View Estates, a development on the bay side of the city between the U. S. Naval Air Station on the west and the ferry landing on the east. The shore is privately owned. The bay shore at Coronado consists of eroding low clay bluffs. No material moves into the area to supply the beach, and the eroding bluffs contribute little material of adequate size to form a protective beach. Property owners desire determination of the most economical means of stabilizing the shore.

The tides on the ocean shore of the study area have a diurnal inequality, the mean and diurnal ranges being respectively about 3.7 and 5.3 feet. In the northern part of San Diego Bay the ranges are 4.2 and 5.8 feet. The maximum tide each year is about 7 feet above mean lower low water. Characteristic waves are swells generated in distant ocean areas. They have heights up to 10 feet and periods up to 20 seconds with the greater heights and shorter periods occurring in the winter, Winter waves generally approach the shore from upcoast of normal, summer waves frequently approach from downcoast of normal. In San Diego Bay waves are those generated by local winds and ship traffic. They have a maximum height of about 2 feet and periods of about 3 seconds and approach the shore from west of normal.

On the ocean shores north of Point Loma at the entrance to San Diego Bay, littoral drift is in general southward in winter and northward in summer. In the Oceanside area southward drift is predominant, as indicated by the general accretion at the Camp Pendleton Harbor jetties and erosion to the south. At Ocean Beach the net annual movement in either direction is negligible. South of Point Loma, the predominant direction of drift is northward. At all ocean front areas there is also a large seasonal onshore-offshore movement of material. At Bay View Estates in Coronado, there is a tendency toward eastward drift along the bay shore, but little beach material is available for transport by wave action.

The Division and District Engineers and the Beach Erosion Board concluded that the most suitable and economical plans of shore protection for the several problem areas are as follows:

a. At Oceanside, construction of a protective beach generally 200 feet wide and approximately 10,000 feet long from the vicinity of Ninth Street to Witherby Street by artificial placement of approximately 900,000 cubic yards of suitable sand;

b. At Ocean Beach, construction of a protective beach generally 20 feet wide and approximately 1,700 feet long between Cape May and Narragansett Avenues by artificial placement of approximately 250,000 cubic yards of suitable sand (at no cost to the project), and construction of one stone groin about 530 feet long in the vicinity of Cape May Avenue;

c. At Imperial Beach, construction of a system of five stone groins from the north end of the existing Naval Radio Station seawall to a point about 400 feet south of Coronado Avenue, the most northerly groin to be 600 feet long and the others each to be about 400 feet long.

d. For Coronado at the Bay View Estates, a stone or broken concrete revetment about 1,900 feet long with a top elevation of 10 feet above mean lower low water.

They concluded that the plans for Oceanside, Ocean Beach and Imperial Beach are justified by prospective benefits and that the public interest involved in protection of public property warrants Federal assistance in accordance with existing policy. They recommended adoption of separate projects by the United States authorizing Federal participation, subject to certain conditions, by contribution of funds in amount of the entire cost of protecting Federally owned shores and one-third of the costs applicable to protection of other publicly owned shores at Oceanside, Ocean Beach and Imperial Beach. As the shore of Bay View Estates is privately owned, they further concluded that no public interest is involved in the proposed work for that area, and recommended that local interests consider adoption, based upon their own determination of economic justification, of a project for construction of revetment.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

Area 9 of the State of Connecticut study comprises the shore of Long Island Sound between the mouth of East River in Guilford and New Haven Harbor. It includes the shores of the towns of Branford and East Haven and portions of the shores of the town of Guilford and city of New Haven, a total shore frontage of about 29 miles. New Haven, at the west end of this shore area, is about 75 miles east of New York City. The shore area is extensively developed for residential use. The permanent population of Guilford, Branford, East Haven and New Haven is about 193,000. There is a summer increase in the population of about 11,500 in Guilford, Branford and East Haven. In Guilford there are two town-owned low sand spits, one of which is developed as a public beach used for recreational purposes. In Branford, the shore at Parker Memorial Park is owned by the town and is used for recreational purposes. The city of New Haven owns the shore at Lighthouse Point Park, Morris Cove Park, Forbes Bluff and Fort Hale Park. The former has a popular recreational beach.

Long Island Sound is a tidal arm of the Atlantic Ocean. Tides are semi-diurnal, the mean range increasing from 5.4 feet at East River to 6.2 feet at Lighthouse Point. The spring ranges are respectively 6.4 and 7.3 feet. The maximum tide of record at New Haven was 13.9 feet above mean low water or 7.7 feet above mean high water. Tides 3 feet or more above mean high water occur about once a year. With a tidal stage of three feet above mean high water, the maximum height of breakers landward of the low water line is about 8 feet. Larger waves can reach the shore only during infrequent higher tides. Ocean swells entering Long Island Sound between Race Point and Little Gull Island may affect littoral processes, but the waves of primary importance are those generated in the sound. Ordinary short storm waves cause littoral movement and offshore loss of beach material. The influence of swells is probably insufficient to cause appreciable return of material from offshore by wave action. Waves which cause the greatest movement of beach material in this study area are those from the west and southwest causing littoral drift to be predominantly northward along shores aligned generally north and south, and eastward along shores aligned generally east and west.

The study area is characterized by rocky headlands and headlands of unconsolidated glacial material, from which wave-built bars or spits have been formed and the landward areas generally have filled and become marshy. The headlands formerly supplied material to the intervening beaches, but are now generally eroded to bed rock or otherwise protected. The supply of material has thus been reduced or eliminated, and consequently the beaches have slowly deteriorated. Groins have been found to be capable of causing minor accretion areas and stabilizing a narrow band along the upper portion of the beach in some sections, but the natural supply of material is insufficient for the formation of adequate protective beaches by groins alone. The building and maintenance of adequate beaches may be accomplished by artificial placement of sand. The rate of loss of fill can be reduced by groins.

The Division Engineer concluded that practicable plans which merit consideration for the protection and improvement of beaches within the study area are as follows:

a. Guilford Point Beach, Guilford - Widening of 125 feet, approximately 400 feet of beach, by direct placement of sand fill and construction of an impermeable groin 300 feet long at the east limit of the fill.

b. Momauguin Beach, East Haven. Widening to a general width of 125 feet in front of cottages, by direct placement of sand fill, approximately 2,200 feet of Momauguin Beach, construction of a 300-foot extension to an existing groin at Bradford Cove and if necessary to reduce excessive losses of the fill, deferred construction of three additional impermeable groins, 350 feet long.

c. Silver Sands Beach, East Haven - Widening to a general width of 125 feet in front of cottages, by direct placement of sand fill, approximately 2,600 feet of Silver Sands Beach with an added widening of up to 50 feet along the westerly 1,400 feet of shore, construction of an impermeable groin 250 feet long at the mouth of Caroline Creek and, if necessary to reduce excessive losses of the fill, deferred construction of four additional impermeable groins, 350 feet long.

d. West Silver Sands Beach, East Haven - Widening to a general width of 125 feet in front of cottages by direct placement of sand fill approximately 2,950 feet of West Silver Sands Beach with an added widening of up to 50 feet along the westerly 1,800 feet of shore, construction of an impermeable groin 200 feet long at the mouth of Caroline Creek and, if necessary to reduce excessive losses of the fill, deferred construction of four additional impermeable groins 350 to 380 feet long.

e. Shell Beach, East Haven - Widening to a general width of 125 feet in front of cottages by direct placement of sand fill, approximately 1,350 feet of Shell Beach and construction of an impermeable groin 350 feet long at the mouth of Morris Creek.

f. Lighthouse Point Park, New Haven - Construction of an impermeable groin 380 feet long at Lighthouse Point.

The Division Engineer found that protection and improvement of Guilford Point, Momauguin, Silver Sands, West Silver Sands Beaches and Lighthouse Point Park were justified by evaluated benefits. However, for Guilford Point Beach, he found that the limited public benefit, other than recreational, would not warrant adoption of a Federal project and that Momauguin, Silver Sands and West Silver Sands Beaches, being privately owned, were ineligible for Federal aid. He recommended that local interests consider adoption of projects for protection and improvement of those beaches, and that protective measures which may be undertaken by local interests, based on their own determination of economic justification, be accomplished in accordance with the methods

proposed in his report. For Lighthouse Point Park he found the nature and amount of public benefits warranted Federal participation, and recommended subject to certain conditions, adoption of a project by the United States authorizing Federal participation by the contribution of Federal funds in an amount equal to one-third of the first cost of construction of an impermeable groin.

The Beach Erosion Board concurred generally in the methods of protection and improvement proposed by the Division Engineer. However, the Board in considering the plan presented by the Division Engineer for Guilford Point Beach, believed that the proposed beach fill would restore publicly owned lands and the proposed groin would assist in retaining the beach fill. Thus the groin would have protective benefits and a public interest. Accordingly the Board believed that that project would be eligible for Federal aid under the policy established by Public Law 727, 79th Congress. Since the beach fill would presumably be placed at no cost to shore protection, as it would be obtained from dredging of an authorized navigation project and apparently depending only on the Federal appropriation for that project, the Board was of the opinion that the public ownership and interest in protecting the restored beach warrant adoption of a Federal project authorizing Federal contribution to the extent of one-third of the cost of the groin.

The Board noted the inclusion by the Division Engineer of a groin at Caroline Creek for initial construction under the plan for Silver Sands Beach. The Board was of the opinion that the orientation of the shore at this location is such that reversals of direction of littoral drift would cause little loss of the beach fill westward into Caroline Creek. Accordingly, the Board believed that the plan for Silver Sands Beach should be modified to provide that all groins should be considered only for deferred construction, pending demonstration of the need therefor.

The Board recommended that projects be adopted by the United States authorizing Federal participation by the contribution of Federal funds in amount equal to one third of the first costs of measures for the protection of the shores at Guilford Point Beach, Guilford (subject to placement of beach fill at this locality at no cost to shore protection) and Lighthouse Point Park, New Haven, Connecticut, under plans comprising construction of an impermeable groin at each location substantially in accordance with the plans presented by the Division Engineer, with such modification thereof as may be considered advisable by the Chief of Engineers.

The Chief of Engineers concurred generally in the views and recommendations of the Beach Erosion Board.

ATLANTIC COAST OF LONG ISLAND, FIRE ISLAND INLET AND SHORE WESTERLY TO JONES INLET, NEW YORK

The problem area, located about 50 miles east of New York City, includes the barrier beach on the south shore of Long Island from Fire Island Inlet to Jones Inlet, also Oak Beach on the north shore of Fire Island Inlet. This frontage is about 15 miles in length, all publicly owned. Oak Beach, Cedar Island Beach and Gilgo Beach, all owned by the town of Babylon, and Gilgo State Park, owned by the Long Island State Park Commission, occupy the easterly 7 miles of shore all in Suffolk County. Tobay Beach, owned by the town of Oyster Bay, and Jones Beach State Park, owned by the Long Island State Park Commission, occupy the remaining 8 miles of shore, all in Nassau County. The study area also includes Fire Island Inlet and the ocean shore east thereof where sand has been impounded by the jetty at Fire Island Inlet. Jones Beach State Park is a world-famous recreational beach, developed at a cost of over $50,000,000. About 8,000,000 people visit Jones Beach annually, over 10 percent of whom come from areas outside New York State. Facilities at the other public beaches are valued at about $350,000. Further improvement of both Jones Beach and Gilgo State Parks is planned by the Long Island State Park Commission when shore restoration and stabilization are assured.

The tides in the study area are semi-diurnal, the mean and spring ranges being respectively about 4.1 and 5.0 feet. The maximum tide of record was 9.5 feet above mean sea level, or about 11.5 feet above mean low water. The area is exposed to waves generated in distant ocean areas. In general they approach the shore from east of a line normal to the beach, thus causing a predominant westward littoral drift, estimated at 450,000 cubic yards per year on the basis of impoundment from 1940 to 1946 at the Fire Island Inlet jetty. The deficiency in supply to the shore west of that inlet has been estimated to have averaged 300,000 cubic yards annually for the period 1939 to 1955.

The District Engineer concluded that the most practicable plan of shore protection for the Fire Island Inlet-Jones Inlet area is a comprehensive plan which comprises bypassing of sand westward across the Fire Island Inlet to a feeder beach and to Oak Beach, restoration of the shore west of the inlet approximately to its 1939 position, possible channel relocation and construction of stabilization works at Oak Beach. Preservation of the stability of the restored shore thereafter would be accomplished by periodic bypassing of sand. The District Engineer also considered an alternative short-range plan which consists of dredging the inlet shoal opposite the western part of Oak Beach in three operations to relieve the pressure of tidal currents against Oak Beach, to provide a deposition area for littoral drift and to obtain fill material for the feeder beach and for Oak Beach. The District Engineer found that the comprehensive plan was amply justified by prospective benefits, and that the public interest involved in protection of public property warrants Federal participation to the extent of 42 percent of the initial costs and 13 percent of the costs of repeated bypassing operations. The

42-percent share of initial costs is based on Public Law 727, 79th Congress. Since 87 percent of the benefits result from beach erosion control, one-third of 87 percent or 29 percent of the costs would be allocated to the United States. The remaining 13 percent of the benefits result from savings in maintenance of an existing Federal navigation project, therefore this portion of the costs would also be allocated to the United States, making a total Federal share of 42 percent. Based on the benefits to the Federal navigation project, 13 percent of the costs of the repeated bypassing operations would also be borne by the United States. The District Engineer recommended adoption of a project by the United States authorizing Federal participation in the comprehensive plan of restoration and protection, subject to certain conditions, by contribution of shares of the costs as outlined above, provided that those allocations be reviewed in the light of future justification and modified as may be appropriate by the Chief of Engineers after completion of the initial phases.

The Division Engineer concluded that the alternative short-range plan would be adequately effective in preserving the shore westerly to Jones Inlet and would provide immediate erosion relief at a minimum cost. He recommended adoption of a project by the United States authorizing Federal participation, subject to certain conditions, to the extent of 42 percent of the first costs of the alternative short-range plan of restoration and protection, with such modification thereof as in the discretion of the Chief of Engineers may be advisable.

The Beach Erosion Board concurred generally in the views and recommendations of the Division Engineer. The Board noted that the part of the comprehensive plan for restoration and protection of Oak Beach, as presented by the District Engineer, included placement on Oak Beach of 1,550,000 cubic yards of sand from the dredging operation in the littoral reservoir at Democrat Point under phase 1 of the plan. Although the District Engineer anticipated some loss of the initial fill in the absence of measures to shift the strong tidal currents away from Oak Beach, he found no low-cost method of holding the fill. He believed placement of additional fill to offset losses to be the most practicable interim solution. The Board concurred in this opinion. However, with a prospect of losing a considerable portion of the material, the Board was of the opinion that no fill should be placed on Oak Beach until the pressure of tidal currents on Oak Beach can be relieved. With respect to the littoral reservoir east of the Democrat Point jetty, the Board felt that there is reasonable doubt that it would function as intended. The possibility exists that the exit cut through the barrier beach would close and prevent the reservoir from acting as an impounding area. In such event, the lagoon would be undesirable in an area which is planned for recreational development by the Long Island State Park Commission.

The Board noted the District Engineer's opinion that the most suitable method of correcting problem conditions would involve restoration of the shore between Fire Island Inlet and Jones Beach approximately to

the position it occupied in 1939. The District Engineer included such restoration in his comprehensive plan. The short-range plan would provide progressive restoration which would not be uniformly effective over the problem area until the fill placed on the feeder in all three operations had been distributed over that area by wave action, estimated to extend over a period of 15 years. The Board recognized that, under this latter plan, erosion may be experienced at some locations within the problem area before material from the feeder beach reaches those locations. However, the cooperating agency stated that it considered immediate shore restoration to be not essential at this time, and that, pending full restoration, it would place such fills as would be necessary to provide interim protection at critical locations by independent local action. Under this condition the Board believed that inclusion of immediate restoration as an item of the project to be less desirable than progressive restoration. The Board was of the opinion that the short-range plan recommended by the Division Engineer is the most feasible plan for immediate accomplishment of the primary objective of stabilizing Oak Beach and the shore westerly to Jones Inlet, pending possible construction of the bypassing plant proposed by the cooperating agency. The short-range plan provides for dredging the inlet shoal opposite the western part of Oak Beach to relieve the pressure of tidal currents against Oak Beach, to provide a deposition area for littoral drift, and to obtain fill material for the feeder beach and for Oak Beach. The plan anticipates three dredging operations over a project life of 15 years involving a total of about 6,000,000 cubic yards of material. Of this material, about 500,000 cubic yards would be placed on Oak Beach in the initial operation, possibly supplemented in subsequent operations. The remainder of the material would be deposited on the feeder beach west of the inlet. Each pipeline dredging operation would be supplemented, if required, by about two months of dredging by hopper dredge.

The Beach Erosion Board stated its opinion that the public interest associated with savings of public lands and improvements, restoration and preservation of public beach areas and recreational benefits to the general public is sufficient to warrant adoption of a project authorizing Federal participation in the cost of restoring and stabilizing the shore from Oak Beach to Jones Inlet. The share of the expense to be borne by the United States should be one-third of the costs applicable to beach erosion control, plus all the economically justified costs applicable to navigation project maintenance, the costs being allocated between beach erosion control and navigation in proportion to the estimated benefits of those features. The Beach Erosion Board recommended adoption by the United States of a project to restore and protect the shore from Oak Beach to Jones Inlet, substantially as recommended by the Division Engineer, with such modifications thereof as in the discretion of the Chief of Engineers may be advisable. Federal participation was recommended by the contribution of Federal funds in an amount presently estimated at 42 percent of the costs of the work. The Board further recommended that, prior to disbursement of Federal funds for the second and third increments of construction under the project, the Federal shares of the costs for those increments be reevaluated, based on then current estimates of

maintenance costs which can be justified for the navigation project and experience with maintenance requirements on that project to that time, provided, however, that the Federal shares shall not exceed 42 percent of the costs of those increments.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

WAIMEA BEACH AND HANAPEPE BAY, KAUAI, T. H.

Waimea Beach and Hanapepe Bay are located about 7 miles apart on the south shore of Kauai, an island about 27 miles in diameter in the Hawaiian group. Agriculture is the basic economic activity on the island. The populations of the villages of Waimea and Hanapepe are respectively about 5,900 and 1,500. At Waimea, the problem area is about 1,240 feet long, of which 624 feet of shore is publicly owned. At Hanapepe Bay, the problem area is about 1,525 feet long, of which about 1,225 feet of shore is publicly owned. Kauai is an island of volcanic origin with rugged terrain. The steep streams bring large quantities of debris to the shore during periods of storm runoff. The problem beaches, located respectively west of Waimea and Hanapepe Rivers, are composed almost entirely of these materials. Beaches east and west of the problem areas are composed principally of coral and shell derived from marine sources. The prevailing winds in the Hawaiian Islands are the northeast trade winds. Waves generated by these winds are refracted before reaching the south shore of Kauai and approach the problem areas from the southeast. Waves from the south are higher but are less frequent.

At Waimea Beach there appears to be a general westward predominance of littoral drift as a result of the wave action from the southeast and southerly swells, however, there has been accretion at the wall at the mouth of Waimea River. Apparently, the problem area about 1,240 feet long, beginning 500 feet west of the river mouth, is a nodal zone from which material is lost in both directions. At Hanapepe the breakwater on the east side of the bay shelters the bay head beach from waves from the southeast. Diffracted waves and southerly swell cause eastward movement of beach material and progressive erosion on the western half of the shore at the head of the bay.

The Division and District Engineers and the Beach Erosion Board concluded that the plan of protecting the publicly owned shores of Waimea Beach and Hanapepe Bay by constructing rubble-mound seawalls is the most practicable of all the methods studied because of its low initial and annual costs. They further concluded that the work is economically justified and that the Federal Government should participate in the improvements of the publicly owned shores by bearing one-third of the initial costs of construction. They recommended adoption of a project for construction of rubble-mound seawalls for the protection of the shore areas at Waimea Beach and Hanapepe Bay, the United States to reimburse local interests to the extent of one-third of the first cost of protection of the publicly owned shores, subject to certain conditions.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

COMPLETED COOPERATIVE BEACH EROSION STUDIES

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	PUBLISHED IN CONG.	FEDERAL PROJECTS RECOMMENDATION	FEDERAL PROJECTS AUTHORIZED BY CONGRESS
ALABAMA					
Perdido Pass (Alabama Pt.)	18 Jun 54	274	84	Unfav.	
CALIFORNIA					
Santa Barbara - Initial	15 Jan 38	552	75	Fav.	
Suppl.	18 Feb 42				
Final	22 May 47	761	80	Unfav.	
Ballona Creek & San Gabriel R. (Partial)	11 May 38			Unfav.	
Orange County	10 Jan 40	637	76	Unfav.	
Coronado Beach	4 Apr 41	636	77	Unfav.	
Long Beach	3 Apr 42			Unfav.	
Mission Beach	4 Nov 42			Unfav.	
Pt. Mugu to San Pedro BW	27 Jun 51	277	83	Fav.	3 Sep 54
Carpinteria to Pt. Mugu	4 Oct 51	29	83	Fav.	3 Sep 54
Oceanside, Ocean Beach, Imperial Beach & Coronado, San Diego County	26 Jul 55	399	84	Fav.	
CONNECTICUT					
Compo Beach, Westport	18 Apr 35	239	74	Unfav.	
Hawk's Nest Beach, Old Lyme	21 Jun 39			Unfav.	
Ash Crk. to Saugatuck R.	29 Apr 49	454	81	Fav.	17 May 50
Hammonasset R. to East R.	29 Apr 49	474	81	Fav.	3 Sep 54
New Haven Hbr. to Housatonic R.	29 Jun 51	203	83	Fav.	3 Sep 54
Conn. R. to Hammonasset R.	28 Dec. 51	514	82	Unfav.	
Pawcatuck R. to Thames R.	31 Mar 52	31	83	Unfav.	
Niantic Bay to Conn. R.	11 Jul 52	84	83	Unfav.	
Housatonic R. to Ash Creek	12 Mar 53	248	83	Fav.	3 Sep 54
East R. to New Haven Hbr.	15 Nov 55	395	84	Fav.	
FLORIDA					
Blind Pass (Boca Ciega)	1 Feb 37	187	75	Unfav.	
Miami Beach	1 Feb 37	169	75	Unfav.	
Hollywood Beach	28 Apr 37	253	75	Unfav.	
Daytona Beach	15 Mar 38	571	75	Unfav.	
Bakers Haulover Inlet	21 May 45	527	79	Unfav.	

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	PUBLISHED IN CONG.	FEDERAL PROJECTS RECOMMEN-DATION	FEDERAL PROJECTS AUTHORIZED BY CONGRESS
	FLORIDA (cont'd)				
Anna Maria & Longboat Keys	12 Feb 47	760	80	Unfav.	
Jupiter Island	13 Feb 47	765	80	Unfav.	
Palm Beach (1)	13 Feb 47	772	80	Fav.	17 May 50
Pinellas County	22 Apr 53	380	83	Fav.	3 Sep 54
	GEORGIA				
St. Simon Island	18 Mar 40	820	76	Unfav.	
	HAWAII				
Waikiki Beach	5 Aug 52	227	83	Fav.	3 Sep 54
Waimea & Hanapepe Bay, Kauai	17 Jan 56			Fav.	
	ILLINOIS				
State of Illinois	8 Jun 50	28	83	Fav.	3 Sep 54
	LOUISIANA				
Grand Isle	28 Jul 36	92	75	Unfav.	
Grand Isle	28 Jun 54	132	84	Unfav.	
	MAINE				
Old Orchard Beach	20 Sep 35			Unfav.	
Saco	2 Mar 56			Unfav.	
	MASSACHUSETTS				
South Shore of Cape Cod (Pt. Gammon to Chatham)	26 Aug 41			Unfav.	
Salisbury Beach	26 Aug 41			Unfav.	
Winthrop Beach	12 Sep 47	764	80	Fav.	17 May 50
Lynn-Nahant Beach	20 Jan 50	134	82	Fav.	3 Sep 54
Revere Beach	12 Jan 50	146	82	Fav.	3 Sep 54
Nantasket Beach	12 Jan 50			Unfav.	
Quincy Shore	2 May 50	145	82	Fav.	3 Sep 54
Plum Island	18 Nov 52	243	83	Unfav.	

(1) A cooperative study of experimental steel sheet pile groins was also made, under which methods of improvement were recommended in an interim report dated 19 Sep 1940. Final report on experimental groins was published in 1948 as Technical Memo. No. 10 of the Beach Erosion Board.

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	CONG.	FEDERAL PROJECTS RECOMMEN-DATION	AUTHORIZED BY CONGRESS
	MISSISSIPPI				
Hancock County	3 Apr 42			Unfav.	
Harrison County - Initial	15 Mar 44				
Harrison County - Suppl.	16 Feb 48	682	80	Fav.	30 Jun 48
	NEW HAMPSHIRE				
Hampton Beach	15 Jul 32			Unfav.	
Hampton Beach	14 Sep 53	325	83	Fav.	3 Sep 54
	NEW JERSEY				
Manasquan Inlet & Adj.Beaches	15 May 36	71	75	Unfav.	
Atlantic City	11 Jul 49	538	81	Fav.	3 Sep 54
Ocean City	15 Apr 52	184	83	Fav.	3 Sep 54
Sandy Hook to Barnegat Inlet	24 Mar 54	361	84	Fav.	
	NEW YORK				
Jacob Riis Park, Long Island	16 Dec 35	397	74	Unfav.	
Orchard Beach, Pelham Bay, Bronx	30 Aug 37	450	75	Unfav.	
Niagara County	27 Jun 42	271	78	Unfav.	
South Shore of Long Island	6 Aug 46			Unfav.	
Selkirk Shores State Park	21 Oct 53	343	83	Fav.	3 Sep 54
Fair Haven Beach State Park	18 Jun 54	134	84	Fav.	
Hamlin Beach State Park	20 Sep 54	138	84	Fav.	
Braddock Bay State Park	15 Apr 55			Unfav.	
Fire Island Inlet to Jones Inlet	10 Feb 56	411	84	Fav.	
	NORTH CAROLINA				
Fort Fisher	10 Nov 31	204	72	Unfav.	
Wrightsville Beach	2 Jan 34	218	73	Unfav.	
Kitty Hawk, Nags Head & Oregon Inlet	1 Mar 35	155	74	Unfav.	
State of North Carolina	22 May 47	763	80	Unfav.	
	OHIO				
Erie County - Vic. of Huron	26 Aug 41	220	79	Unfav.	
Michigan Line to Marblehead	30 Oct 44	177	79	Unfav.	

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	PUBLISHED IN CONG.	FEDERAL PROJECTS RECOMMEN-DATION	FEDERAL PROJECTS AUTHORIZED BY CONGRESS
OHIO (cont'd)					
Cities of Cleveland & Lakewood	22 Mar 48	502	81	Fav.	3 Sep 54
Chagrin River to Fairport	22 Nov 49	596	81	Unfav.	
Vermilion to Sheffield Lake Village	24 Jul 50	229	83	Fav.	3 Sep 54
Fairport to Ashtabula	1 Aug 51	351	82	Unfav.	
Ashtabula to Penna. St. Line	1 Aug 51	350	82	Unfav.	
Sandusky to Vermilion	7 Jul 52	32	83	Unfav.	
Sandusky Bay	31 Oct 52	126	83	Unfav.	
Sheffield Lake V. to Rocky R.	31 Oct 52	127	83	Unfav.	
Euclid to Chagrin River	25 Jun 53	324	83	Unfav.	
PENNSYLVANIA					
Presque Isle Peninsula, Erie (Interim)	3 Apr 42				
(Final)	23 Apr 52	231	83	Fav.	3 Sep 54
PUERTO RICO					
Punta Las Marias, San Juan	5 Aug 47	769	80	Unfav.	
RHODE ISLAND					
South Shore (Towns of Narragansett, South Kingstown, Charlestown & Westerly)	4 Dec 48	490	81	Fav.	3 Sep 54
SOUTH CAROLINA					
Folly Beach	31 Jan 35	156	74	Unfav.	
Pawleys Is., Edisto Beach & Hunting Island	24 Jul 51			Unfav.	
TEXAS					
Galveston (Gulf Shore)	10 May 34	400	73	Unfav.	
Galveston Bay, Harris County	31 Jul 34	74	74	Unfav.	
Galveston (Gulf Shore)	5 Feb 53	218	83	Unfav.	
Galveston (Bay Shore)	19 Jun 53	346	83	Unfav.	

LOCATION	BEB REPORT COMPLETED	PUBLSIHED IN H. DOC.	CONG.	FEDERAL PROJECTS RECOMMEN-DATION	AUTHORIZED BY CONGRESS
	VIRGINIA				
Willoughby Spit, Norfolk	20 Nov 37	482	75	Unfav.	
Colonial Beach, Potomac River	24 Jan 49	333	81	Fav.	17 May 50
Virginia Beach	25 Jun 52	186	83	Fav.	3 Sep 54
	WISCONSIN				
Milwaukee County	21 May 45	526	79	Unfav.	
Racine County	5 Mar 52	88	83	Unfav.	
Kenosha	16 Sep 54	273	84	Unfav.	
Manitowoc County	15 Apr 55	348	84	Fav.	

AUTHORIZED COOPERATIVE BEACH EROSION STUDIES

CALIFORNIA

STATE OF CALIFORNIA. Cooperating Agency: Department of Public Works, Division of Water Resources, State of California

Problem: To conduct a study of the problems of beach erosion and shore protection along the entire coast of California. The current studies cover the Santa Cruz, Orange County, San Diego County and Humboldt Bay Areas.

CONNECTICUT

STATE OF CONNECTICUT. Cooperating Agency: State of Connecticut (Acting through the Flood Control and Water Policy Commission)

Problem: To determine the most suitable methods of stabilizing and improving the shore line. Sections of the coast are being studied in order of priority as requested by the cooperating agency until the entire coast has been included. Study areas currently remaining for completion include the shores between the Saugatuck and Byram Rivers and between the Thames and Niantic Rivers.

DELAWARE

STATE OF DELAWARE. Cooperating Agency: State Highway Department

Problem: To formulate a comprehensive plan for restoration of adequate protective and recreational beaches and a program for providing continued stability of the shores from Kitts Hummock on Delaware Bay to Fenwick Island on the Atlantic Ocean.

FLORIDA

PALM BEACH COUNTY. Cooperating Agency: Board of County Commissioners, Palm Beach County.

Problem: To develop the most economical means of restoring the beaches between Lake Worth Inlet and South Lake Worth Inlet to a satisfactory condition and protecting the restored beaches and shore property from future erosion.

MASSACHUSETTS

PEMBERTON POINT TO GURNET POINT. Cooperating Agency: Department of Public Works.

Problem: To determine the most suitable methods of shore protection, prevention of further erosion and improvement of beaches, and specifically to develop plans for protection of Crescent Beach, the Glades, North Scituate Beach and Brant Rock.

CHATHAM. Cooperating Agency: Department of Public Works.

Problem: To determine the best method of preventing shoaling of Stage Harbor and damage to shore property, and the effects on Stage Harbor and adjacent shore property of probable changes to Nauset Beach and Monomoy Island and any works which may be constructed for protection of Stage Harbor.

MICHIGAN

BERRIEN COUNTY. Cooperating Agency: City of St. Joseph.

Problem: To determine the most effective methods of preventing erosion of the shore by waves and currents.

NEW JERSEY

STATE OF NEW JERSEY. Cooperating Agency: Department of Conservation and Economic Development.

Problem: To determine the best method of preventing further erosion and stabilizing and restoring the beaches, to recommend remedial measures, and to formulate a comprehensive plan for beach preservation or coastal protection. Current studies cover the Atlantic Ocean shore from Barnegat Inlet to Cape May Canal, Cape May Canal to Maurice River in Delaware Bay, and South Amboy to Shrewsbury River in Raritan and Sandy Hook Bays.

NEW YORK

SUFFOLK COUNTY (ATLANTIC COAST BETWEEN MONTAUK POINT AND FIRE ISLAND INLET). Cooperating Agency: Department of Public Works, State of New York.

Problem: To determine the most practicable and economic method of restoring adequate recreational and protective beaches and providing continued stability to the shores.

NORTH CAROLINA

CAROLINA BEACH. Cooperating Agency: Town of Carolina Beach.

Problem: To determine the best method of preventing erosion of the beach.

OHIO

MICHIGAN LINE TO MARBLEHEAD. Cooperating Agency: Division of Shore Erosion, Department of Natural Resources, State of Ohio

Problem: To determine the best method of protecting the shores of the study area, including typical methods of protection for publicly and privately owned shores, especially to determine whether any changes should be made in recommendations contained in H.D. No. 177, 79th Congress in view of changed conditions and additional data; and to develop specific plans of restoration and protection of the shores of Metzgar Marsh, Crane Creek State Park and East Harbor State Park, and general plans for the protection of privately owned property.

RHODE ISLAND

SOUTH KINGSTOWN AND WESTERLY. Cooperating Agency: Division of Harbors and Rivers, Department of Public Works, State of Rhode Island

Problem: To develop the most suitable method of restoring and stabilizing the shores in connection with development of public beaches at East Matunuck in South Kingstown and Misquamicut Beach in Westerly.